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Design of a  
Reinforced-Concrete Viaduct

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DESIGN OF A REINFORCED-CONCRETE  
VIADUCT

BY

GEORGE HAROLD SMITH

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THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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COLLEGE OF ENGINEERING

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June 1, 1910

This is to certify that the thesis of GEORGE HAROLD SMITH entitled Design of a Reinforced Concrete Viaduct is approved by me as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

F. O. Sufour

Instructor in Charge.

Approved:

Ira O. Baker.

Professor of Civil Engineering.





## DESIGN OF A REINFORCED CONCRETE VIADUCT

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## DESIGN OF A REINFORCED CONCRETE VIADUCT.

### I. INTRODUCTION

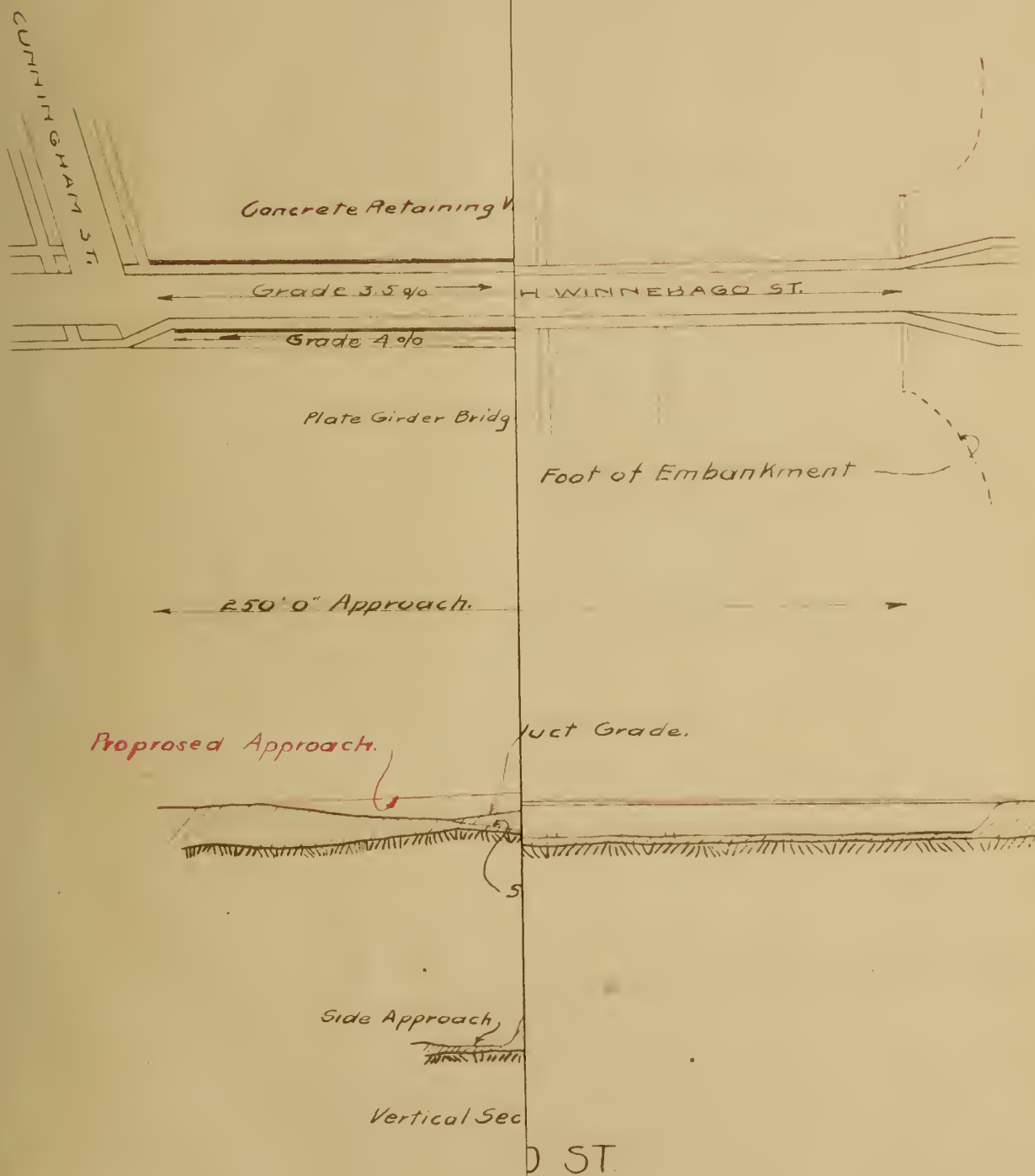
The city of Rockford is at the present time contemplating the erection of a new viaduct. The proposed structure will carry the traffic of South Winnebago Street across the flat valley through which run two small creeks and eleven tracks of the "Illinois Central" and the "Chicago, Milwaukee, and St. Paul", railroads.

The site will require the erection of a structure about nine hundred feet in length. The north bluff of the valley is slightly below the viaduct grade. The south bluff is about ten feet below grade. Plate I shows the profile and plan of the site. In order to bring the street grade up to the viaduct grade an approach of about two hundred and fifty feet in length will be necessary. A narrow roadway will be required from the street proper to the level of the tracks in order to accommodate the trucking done in the valley. All of the tracks are at about the same grade, those at the south of the valley being about one foot higher than those at the north side. The surface of the water in the creeks is approximately eight feet below the track levels. Limestone underlies the valley at a depth of three to eight feet below the ground surface.

The present structure is, with the exception of a short truss span, a series of timber bents supported by planks resting upon the ground surface. The timber truss-span, carrying







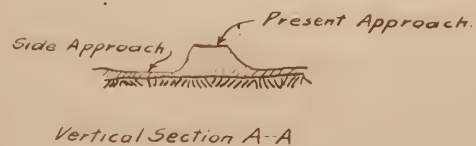
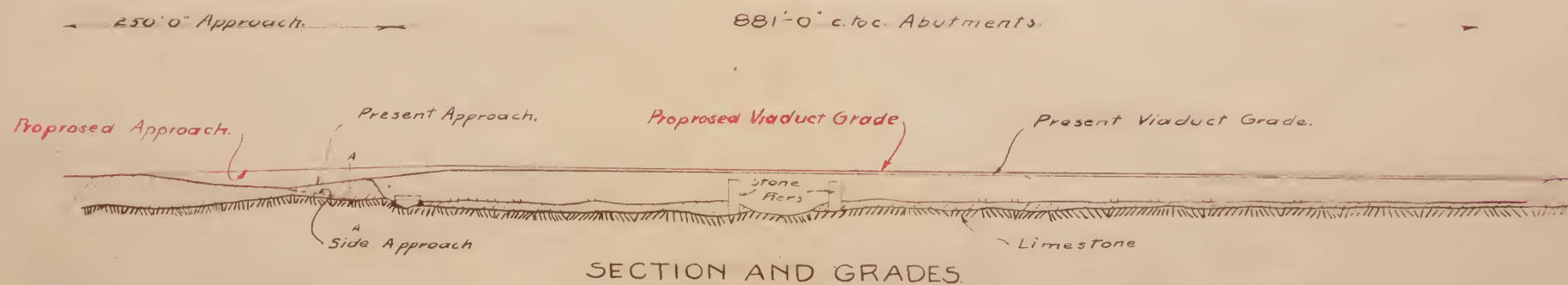
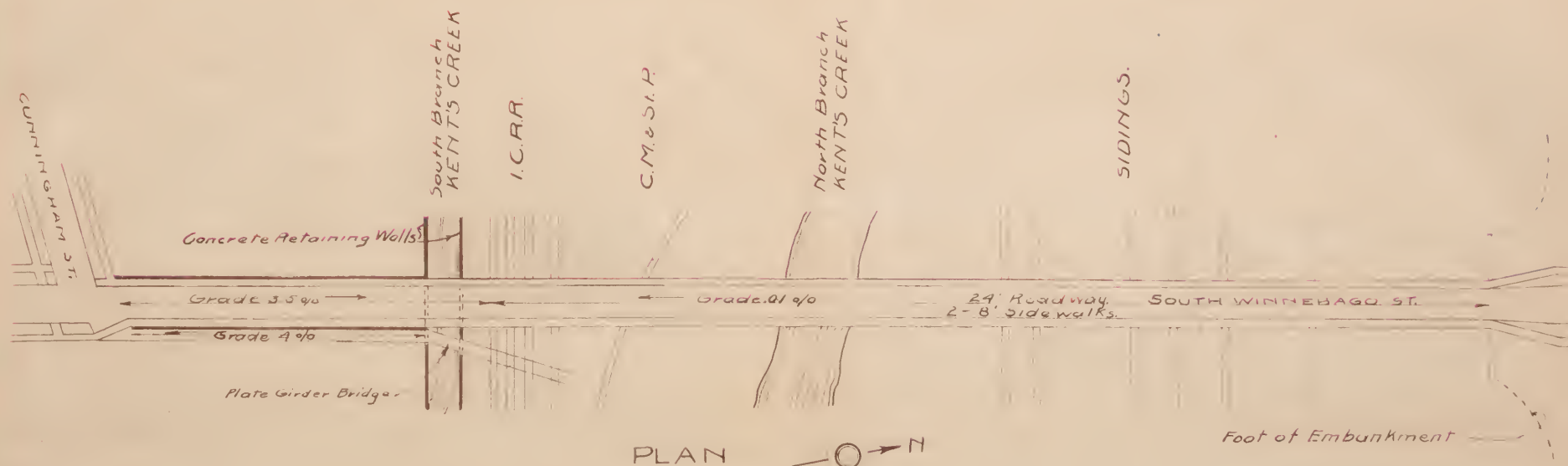


PLATE I  
 PLAN AND GRADES  
 OF  
 THE SOUTH WINNEBAGO ST  
 VIADUCT  
 ROCKFORD ILLINOIS

the floor system across the north branch of the creek, is supported upon two rough stone piers. The south branch of the creek flows thru a 32-foot channel, the banks of which are retained by concrete walls.

## II. DETERMINATION OF TYPE OF STRUCTURE.

The viaduct being across a freight yard will be exposed to the gases from the locomotives. Because of the inability of steel to withstand the destructive influence of these agents, concrete construction was decided upon as the material for the new structure. The type chosen is a flat, combined beam-and-slab girder supported upon flat arch piers. Fig. 1. shows a side elevation and section of the floor system and piers. Non-continuity of the entire floor system, over the piers will be maintained. The spans will be uniform except at the north end of the structure where it will be 35 feet, the standard length being 47 feet. The span of 47 feet is chosen because it will span three tracks placed upon 14-foot centers and allow for a pier five feet thick.

The roadway will be 24 feet in width with two 8-foot sidewalks. The railings will be concrete slabs, the ends of which will rest in recesses in the posts. To prevent a rigid union between the post and slab, a heavy coating of asphalt will be applied to the interior of the recesses before the slabs are cast. Fig. 2. shows an elevation and section of the railing. Ornamental lamp posts will be set up over each pier. These will be secured to the rail post top by small anchor bolts imbedded in





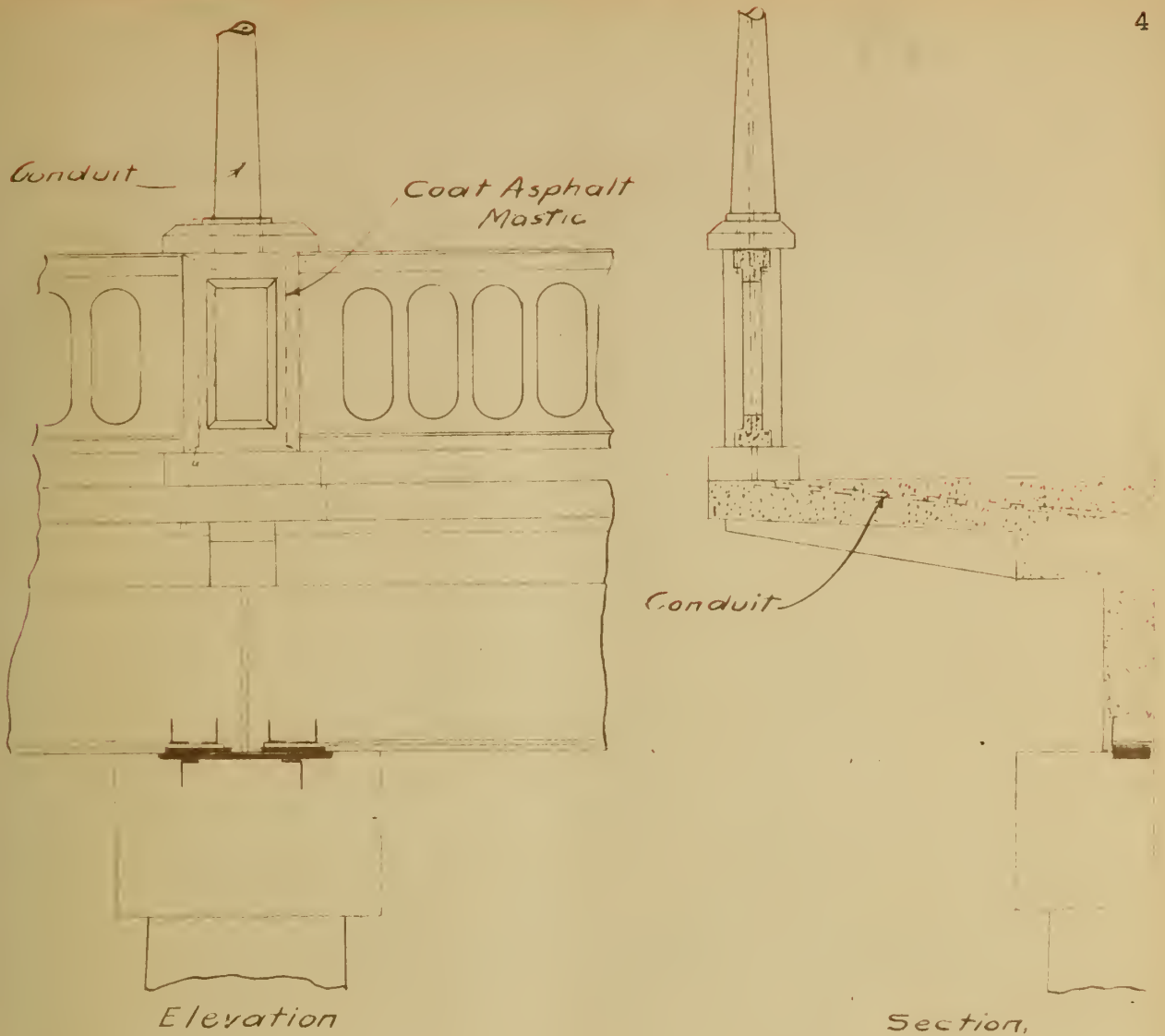


Fig 2 Elevation and Section of Railing

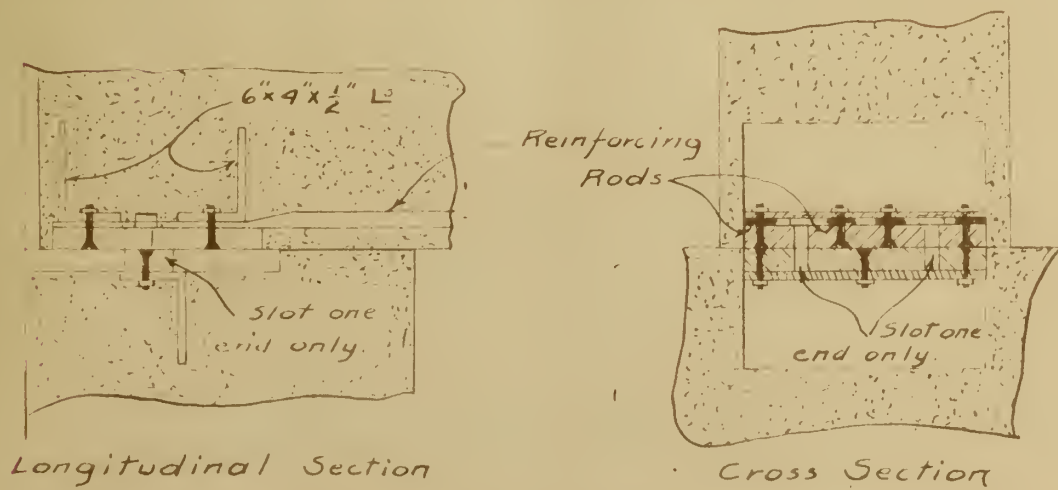


Fig. 3. Longitudinal and Cross Sections of Cirders showing Bearing Plates.



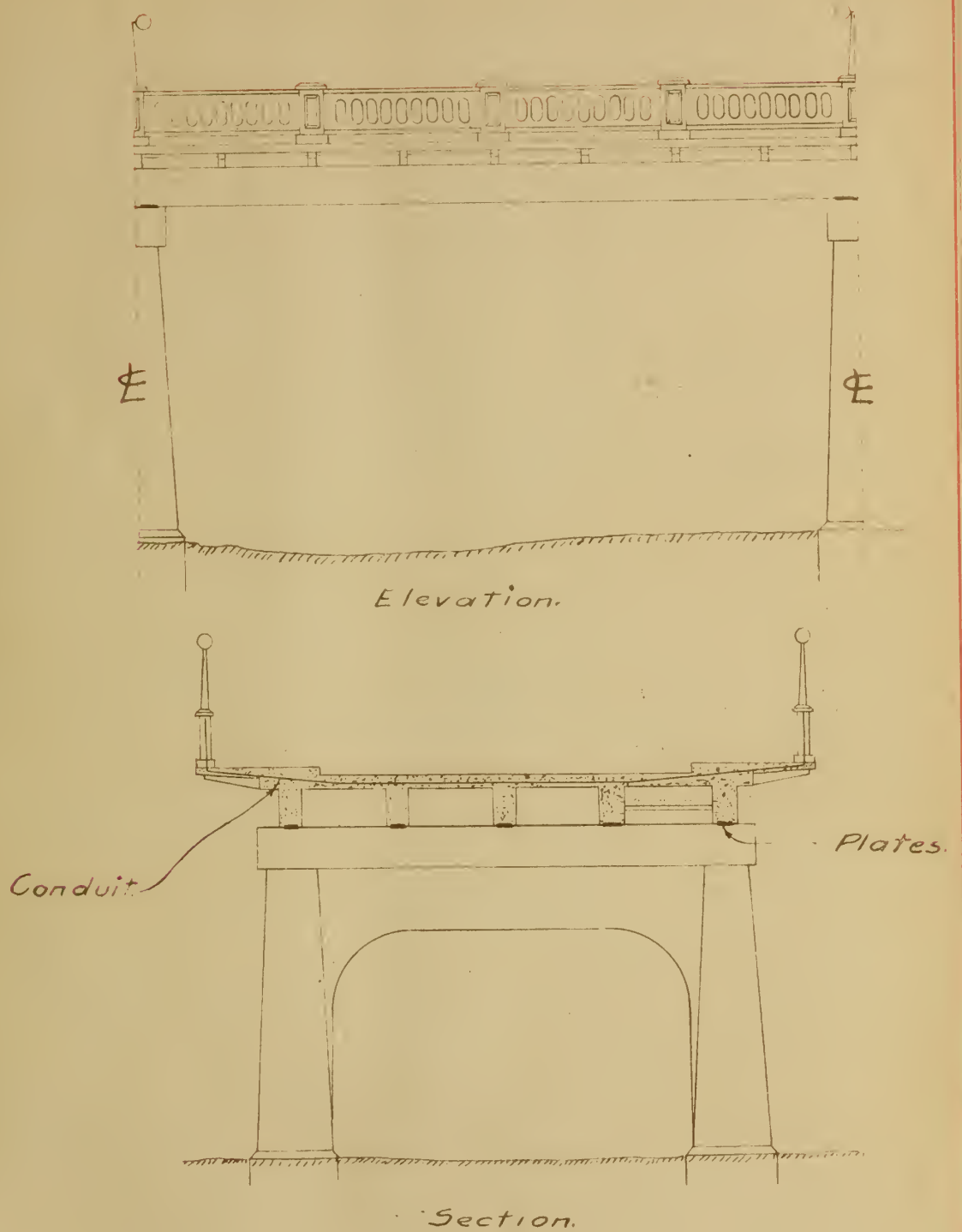


Fig.1. Typical Girder Span and Floor Section.





the concrete. To eliminate the presence of telephone and electric light wires over the street way, provision will be made to carry them underneath the floor system upon wooden cross beams between two girders as shown in Fig. 1. A small conduit pipe will be imbedded in the walk slab leading from the base of the light post to the open wire-conduit beneath the floor.

The piers, as shown in Fig. 1., will be flat arches supported upon massive columns. Cast iron plates set in the piers and corresponding ones in the bottoms of the girders will allow for expansion. See Fig. 3.

The abutments are shown in Figs. 4 and 5. The counter-foresides will be directly the girder seats. The wings of the south abutment will be turned back at 90 degrees and continued as retaining walls. The north abutment will be straight. See Fig. 5. The retaining walls are shown in section and elevation in Fig. 4.

The pavement will be of wooden block. A two inch crown will be secured by a cushion of sand varying in thickness from  $1\frac{1}{2}$  inch at the gutter to 2 inches at the center of the roadway. In order to provide a sufficient slope so that the roadway will drain, the concrete slab is given a crown of  $1\frac{1}{2}$  inch and a longitudinal grade of five one hundredths of a foot per span. To prevent any water which may drain through the paving from seeping into the concrete and coming in contact with the steel, a water proofing will be given to the top of the floor slab. This will consist of a double layer of burlap and asphalt mastic. The weep holes will be near the lower ends of the gutters as shown



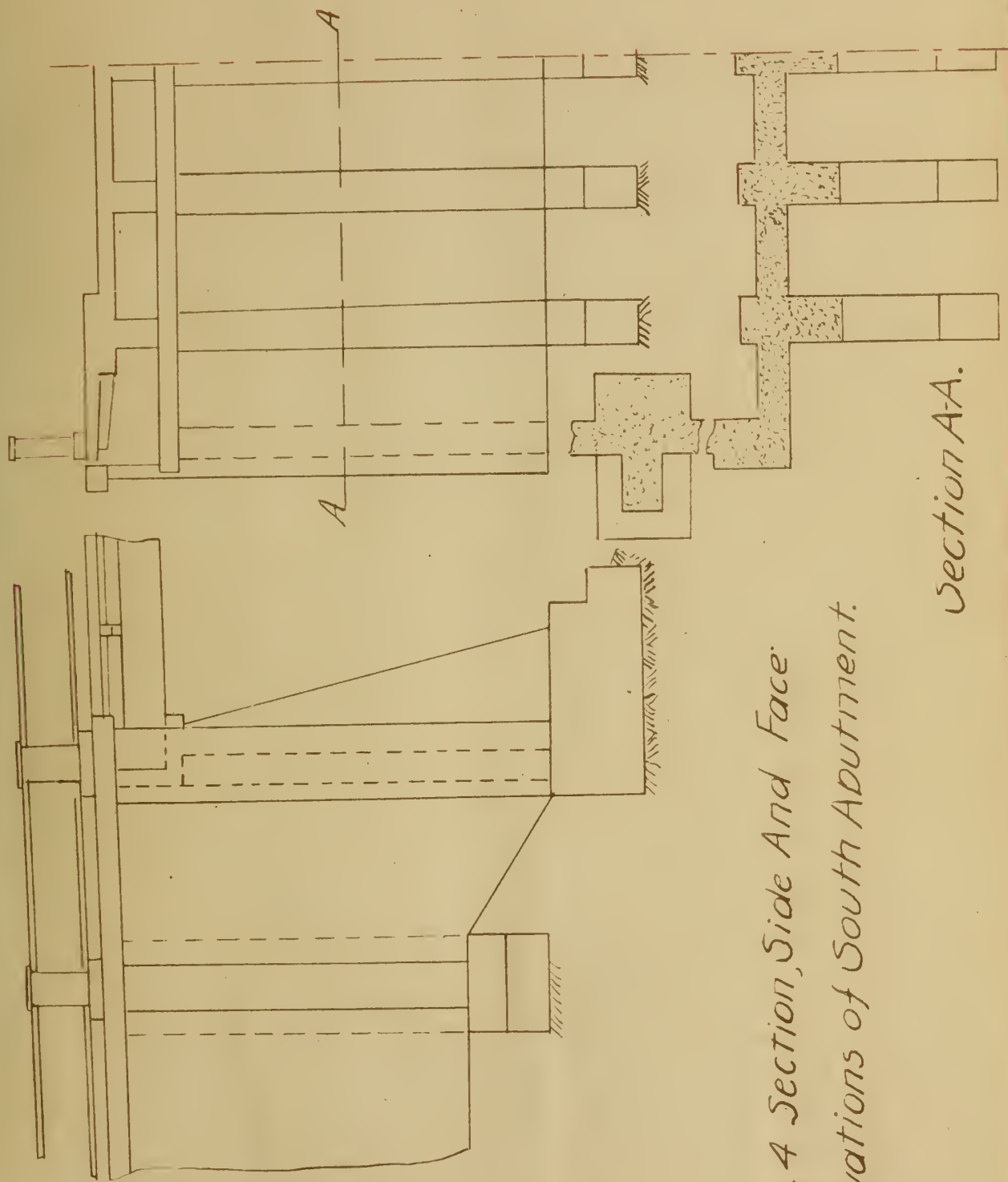
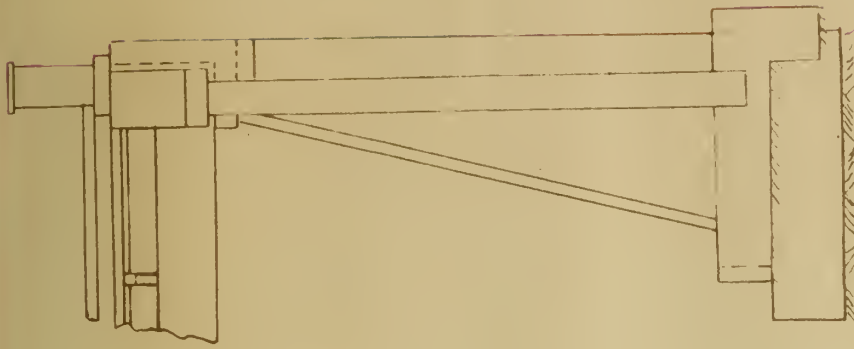


Fig. 4 Section, Side And Face.  
Elevations of South Abutment.

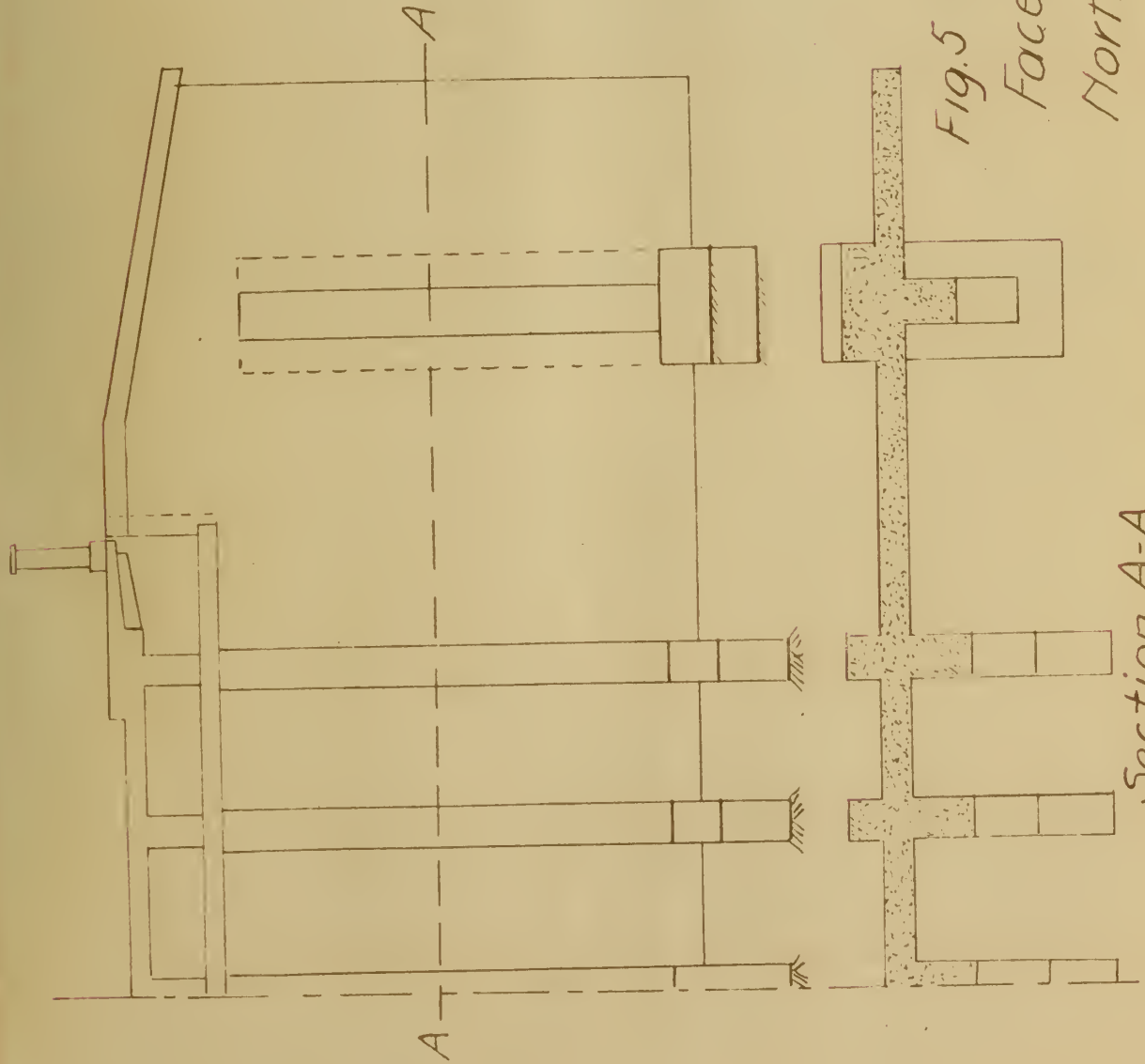
Section A-A.







*Fig. 5 Section, Side And  
Face Elevations of  
North Abutment.*



*Section A-A.*



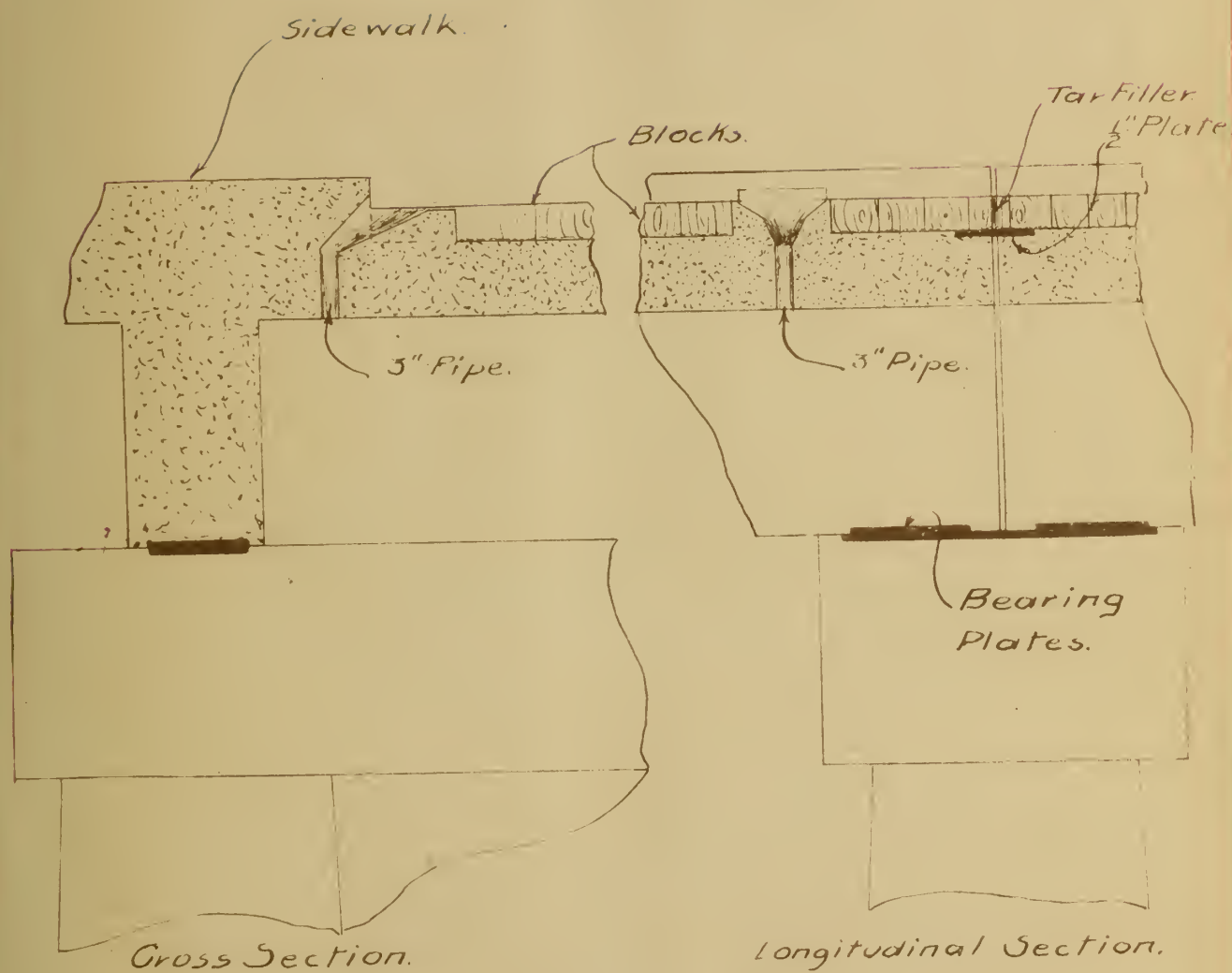


Fig. 6. Longitudinal and Cross Sections of Gutter showing Weep holes and Girder Joints.



in Fig. 6. The girder ends will be notched at the top, to allow a steel plate to be dropped in to carry the blocks across the opening. This detail is shown in Fig. 6.

### III. PRINCIPLES GOVERNING THE DESIGN.

In the design of the structure several essential features must be kept in mind. Simplicity and uniformity are essential to economical construction from the builder's point of view. Elaborate details necessitate expensive forms. Uniformity will allow sectional forms to be used several times thus decreasing the cost of forms and also a saving of time. The depth of the girders must also be kept at a minimum since any increase in this dimension will increase the grade of the approaches.

### IV. SPECIFICATIONS.

The general specifications governing the design are as follows: The loadings will be taken large enough to provide for any increase in the traffic over that of the present time. The allowable stresses, loadings, and mixtures are such as conform with present practice. For convenience these are given in Tables I, II, and III.





TABLE I

## Allowable Stresses

Stress	Pounds per Square Inch
Compression of concrete in flexure - - - - -	600
Shear in plain concrete - - - - -	30
Shear in tensile reinforced concrete - - - -	100
Bearing of plates on concrete - - - - -	400
Bond strength - - - - -	75
Tension of steel in flexure - - - - -	14,000
Tension of steel in flexure and temperature	16,000

TABLE II

## Loadings

Dead Load	
Weight concrete	150 pounds per cubic foot.
Weight pavement	50 pounds per square foot.
Live Load	
200 pounds per square foot or:	
24 ton roller: 18 tons on rear axle.	
Axles spaced on 10 foot centers.	

TABLE III

## Proportions for Concrete

Part of Structure	Parts Cement	Sand	Stone
Floor system	1	2	4
Footings	1	3	6
Piers	1	3	6
Walls	1	3	6
Railings and Posts	1	4	0



For the floor system, the stone shall be under 2 inches in diameter. For the footings and piers, stones up to one man size may be used.

The steel shall have an elastic limit of 35,000 pounds per square inch. Square twisted bars are to be used throughout.

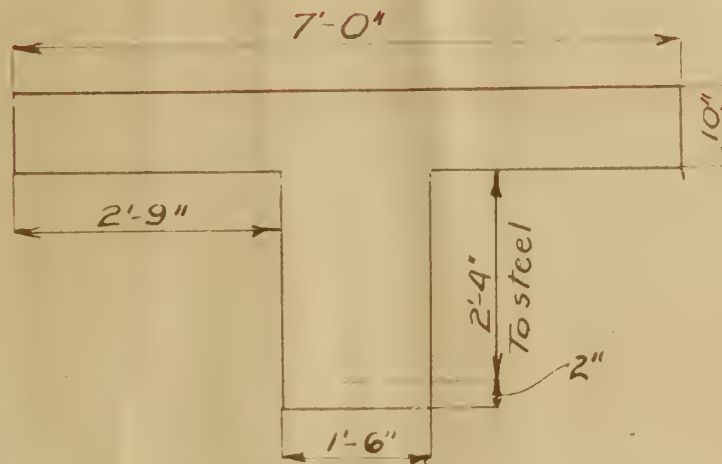
The theory and nomenclature used is that given by Turneaure and Maurer in the 1909 edition of their "Principles of Reinforced Concrete Construction". Any reference to formulae or diagrams refer to this text unless otherwise stated. The diagrams are used in preliminary computations, and these computations are not recorded. The following computations are for the final selections.

#### V. DESIGN OF THE FLOOR.

The first member to be designed is the longitudinal floor-slab girders. The section as shown in Fig. 1. may be divided into a series of tee-beams. The three central girders are the same in cross-section. The design will be made for a tee-beam considering the compression stress as being transmitted over a width of floor slab equal to the girder spacings. Fig. 7 shows the dimensions of the beam. This section conforms with the recommendations given in the text in the following manner, inasmuch as the projecting flange is less than four times the width of the stem, and the width of the stem as chosen is determined by the shear which is to be carried.







*Fig. 7. Cross Section of Tee Beam.*

The loading which this beam carries consists of its own weight, the weight of the paving, and either the uniform live load or the roller load as given in Table II. The constant dead load per lineal foot will be determined first.

The area of the section as shown is 1,416 square inches or 9.83 square feet. The load per lineal foot is:

Concrete	$9.83 \times 150 =$	1,475 pounds
Paving	$7.00 \times 50 =$	<u>350 pounds</u>
Total		$= 1,825 \text{ pounds.}$

The total live load per foot is:

$$7.00 \times 200 = 1,400 \text{ pounds.}$$

The total uniform load per foot is:

Dead load	1,825 pounds.
Live load	<u>1,400 pounds.</u>
Total	3,225 pounds.



Since the length of the span is 47 feet, the shear at the end of the girder is:

$$3,225 \times \frac{47}{8} = 75,750 \text{ pounds.}$$

The area taking shear is 18 inches by 40 inches, and the unit shear is therefore;

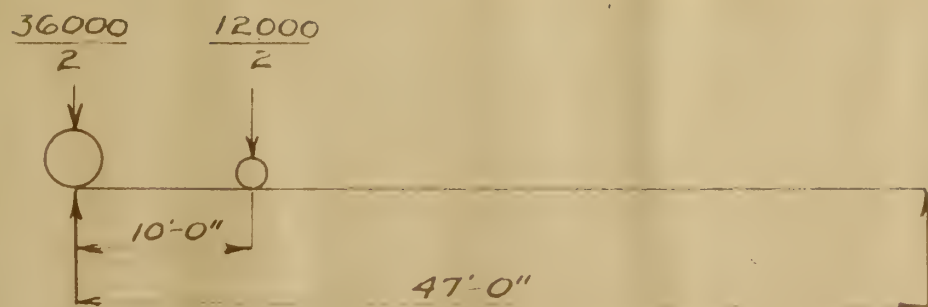
$$\frac{75,750}{18 \times 40} = 100 \text{ pounds per square inch. This is}$$

allowable. See Table I.

The maximum moment at the center of the beam due to uniform load is:

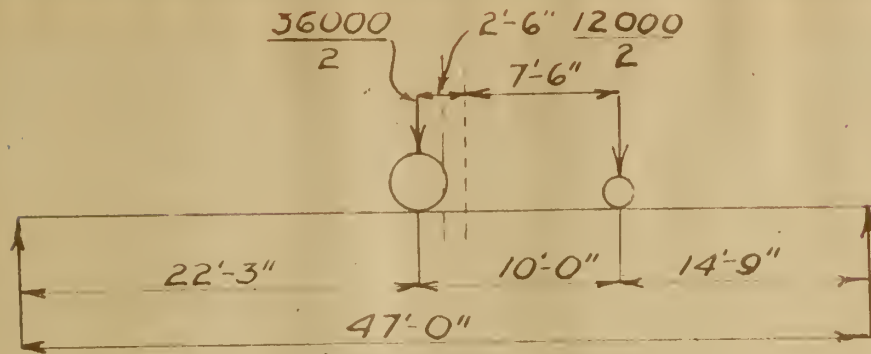
$$\frac{3,225 \times 47 \times 47 \times 12}{8} = 10,670,000 \text{ pound-inches.}$$

The maximum shear and moment resulting from the roller load will now be determined. The load of 24 tons is assumed to be carried by two adjacent beams. The maximum shear will be produced with the roller placed as in Fig. 8. The maximum moment will be produced with the roller as shown in Fig. 9. The position for maximum moment is reached when the rear axle is as far from one side of the center of the span as the center of gravity of the loads on the two axles is on the other side.



*Fig. 8. Position of Roller For Maximum Shear.*





*Fig. 9. Position of Roller For Maximum Moment.*

The total end shear for the roller placed as in Fig. 8 is:

$$\text{Dead load } 1,825 \times \frac{47}{2} = 42,900 \text{ pounds.}$$

$$\text{Roller load } \frac{36,000}{2} \frac{12,000}{2} \frac{37}{47} = 22,720 \text{ pounds.}$$

$$\text{Total - - - - -} = 65,620 \text{ pounds.}$$

As this value is less than 75,750 pounds resulting from the uniform load, the section under shear is not over-stressed.

The total moment with the roller placed as in Fig. 9 is:

$$\text{Dead load } \frac{1,825 \times 47 \times 47 \times 12}{8} \text{ Pound-inches} = 6,040,000$$

$$\text{Roller load } \frac{18,000 \times 24.75}{47} + \frac{6,000 \times 14.75}{47} \quad 22.25 \times 12 = 3,030,000$$

$$\text{Total - - - - -} = 9,070,000$$

This moment resulting from the roller placed as in Fig. 9 is less than the moment resulting from the uniform load.

The area of the steel required to carry the maximum moment of 10,670,000 pound-inches is computed from the formulae:

$$M_s = F_s A_s d$$





where,

$M_s$  = the total moment to be carried,

$F_s$  = the allowable stress per unit area of the steel,

$A$  = the total area of the steel in square inches,

$d$  = the depth of the steel below the top of the slab, and

$j$  = the ratio of the resisting couple to  $d$ . See Fig. 10.

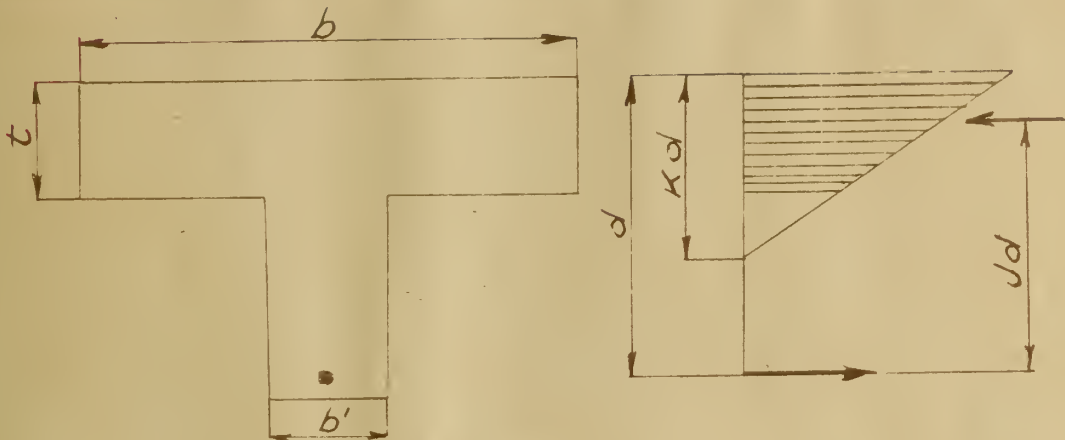


Fig. 10.

From the diagram on p. 280 the value of  $j$  is approximately 0.9 for all Tee-beams in which  $\frac{t}{d}$  is (.25). This value of  $j$  will be assumed for the present and changed later if necessary. By substituting in the above formulae the area of the steel required is:

$$A = \frac{10,670,000}{0.9 \times 40 \times 1400}$$

$$= 21.2 \text{ square inches.}$$

The percent of reinforcement is:

$$\frac{21.2}{40 \times 84} = .631$$



The true value of  $j$  must now be determined. From p. 85:

$$j = \frac{6-6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 - \left(\frac{t}{d}\right)^3}{6-3\left(\frac{t}{d}\right)} / pn^2$$

where,  $j$  is the moment arm of the couple,  
 $t$  is the thickness of the slab in inches,  
 $d$  is the effective depth of the beam in inches,  
 $p$  is the percentage of reinforcement, and  
 $n$  is the ratio of the modulus of elasticity of steel  
to that of concrete. See Fig. 10.

Substituting and solving:

$$j = \frac{6-6\left(\frac{10}{40}\right) + 2\left(\frac{10}{40}\right)^2 - \left(\frac{10}{40}\right)^3}{6-3\left(\frac{10}{40}\right)} / 15 \times 0.00631 \times 2$$

$$= 0.88$$

The assumed value of  $j$  was 0.9. Resubstituting the value of 0.88 in place of 0.9, the value of  $A$  is found to be 21.6 square inches. The percentage of steel with this reinforcement is:

$$\frac{21.6}{40 \times 84} = .643$$

Computing  $j$  from this percentage we have 0.88 which checks with the second trial value of  $j$ .

The maximum compression upon the concrete is found from the formulæ on p. 86, and is:

$$Mc = Fc\left(1 - \frac{t}{2kd}\right) b.t.j.d.$$





where,  $M_G$  = the maximum moment to be carried by the concrete,  
 $F_c$  = the unit compressive stress allowed in the concrete,  
 $t$  = the thickness of the slab in inches,  
 $K$  = the depth of the neutral axis below the slab top  
 (see Fig. 10)  
 $d$  = the effective depth of the beam,  
 $b$  = the breadth of the beam, and  
 $j$  = the moment arm of the couple.

Before substituting in this formulae, the value of  $K$  must be found from the formulae on p. 85:

$$K = \frac{Pn + \frac{1}{2} \left( \frac{t}{d} \right)^2}{Pn + \left( \frac{t}{d} \right)}$$

where,  $p$  is the percent of reinforcement,  
 $n$  is the ratio of the modulus of elasticity of the concrete to that of steel,  
 $t$  is the thickness of the slab in inches, and  
 $d$  is the effective depth of the beam.

Substituting in the above formulae the value of  $K$  is found to be:

$$\begin{aligned} K &= \frac{15 \times 0.00643 \times \frac{1}{2} \left( \frac{10}{40} \right)^2}{15 \times 0.00643 + \left( \frac{10}{40} \right)} \\ &= 0.368 \end{aligned}$$

Substituting in the formulae for the unit compressive stress upon the concrete and using this value of  $K$ , there results:

$$10,670,000 = F_c \left( 1 - \frac{10}{2 \times 0.368 \times 40} \right) 84 \times 10 \times 0.88 \quad \left( \begin{array}{l} \times 40 \\ \times 0.88 \end{array} \right)$$



$$x = \sqrt{\frac{47}{22.04}} \sqrt{4.50}$$

$$= 21.1 \text{ feet.}$$

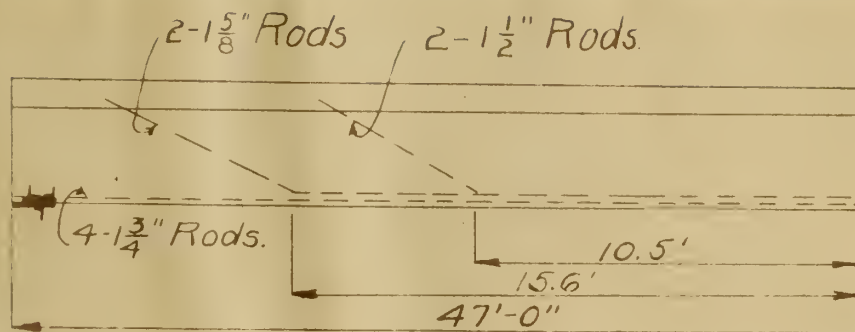
The two 1 1/2-inch rods may be turned up at a distance of 10 1/2 feet from the center.

The total length of the 1 5/8-inch rods is:

$$x = \sqrt{\frac{47}{22.04}} \sqrt{5.28 + 4.50}$$

$$= 31.3 \text{ feet.}$$

The two 1 5/8 inch rods may be turned up at a distance of 15.6 feet from the center. The arrangement of the rods is shown in Fig. 11.



*Fig. 11. Position of Bent Up Rods in Central Girders*

The bond stress which must exist for the four rods running straight is:

$$U = \frac{V}{Jd}$$

where  $U$  = the total bond stress developed per lineal inch,  
 $V$  = total shear at the end,  
 $J$  = the moment arm of the resultant couple, and  
 $d$  = the depth of the beam.



$F_c = 548$  pounds per square inch for the unit compressive stress, which value is less than the stress of 600 pounds allowable. (see Table I).

The steel area of 21.6 square inches necessary to carry the maximum moment at the center is made up as follows:

$$4 - 1 \frac{3}{4} \text{ inch rods} = 4 \times 3.0625 = 12.26 \text{ square inches}$$

$$2 - 1 \frac{5}{8} \text{ inch rods} = 2 \times 2.6400 = 5.28 \text{ square inches}$$

$$2 - 1 \frac{1}{2} \text{ inch rods} = 2 \times 2.2500 = \underline{4.50 \text{ square inches}}$$

$$\text{Total area of rods} \quad 22.04 \text{ square inches.}$$

This is a little in excess of the required area of steel but will over-balance any possible under-sized rods.

As the moment varies from maximum at the center to zero over the supports, part of the rods can be turned up into the web. The length of the rods is computed from the formulae, p. 221, which is:

$$X_n = \sqrt{\frac{1}{A}} \sqrt{a_1 + a_2 + \dots + a_n}$$

where,

$X_n$  = the length of the nth rod in order of length, counting the shortest as number one;

$l$  = length of span in feet;

$A$  = total steel area in square inches at center; and

$a_1 \dots a_n$  = area of each rod up to the nth.

The 4 - 1 3/4 inch rods will be carried entirely through the girder bottom. The other 4 rods will be turned up in pairs.

The total length of the 1 1/2 inch rods is:





Substituting in the above formulae:

$$U = \frac{75,750}{0.88 \times 40} = 2,130 \text{ pounds per lineal inch.}$$

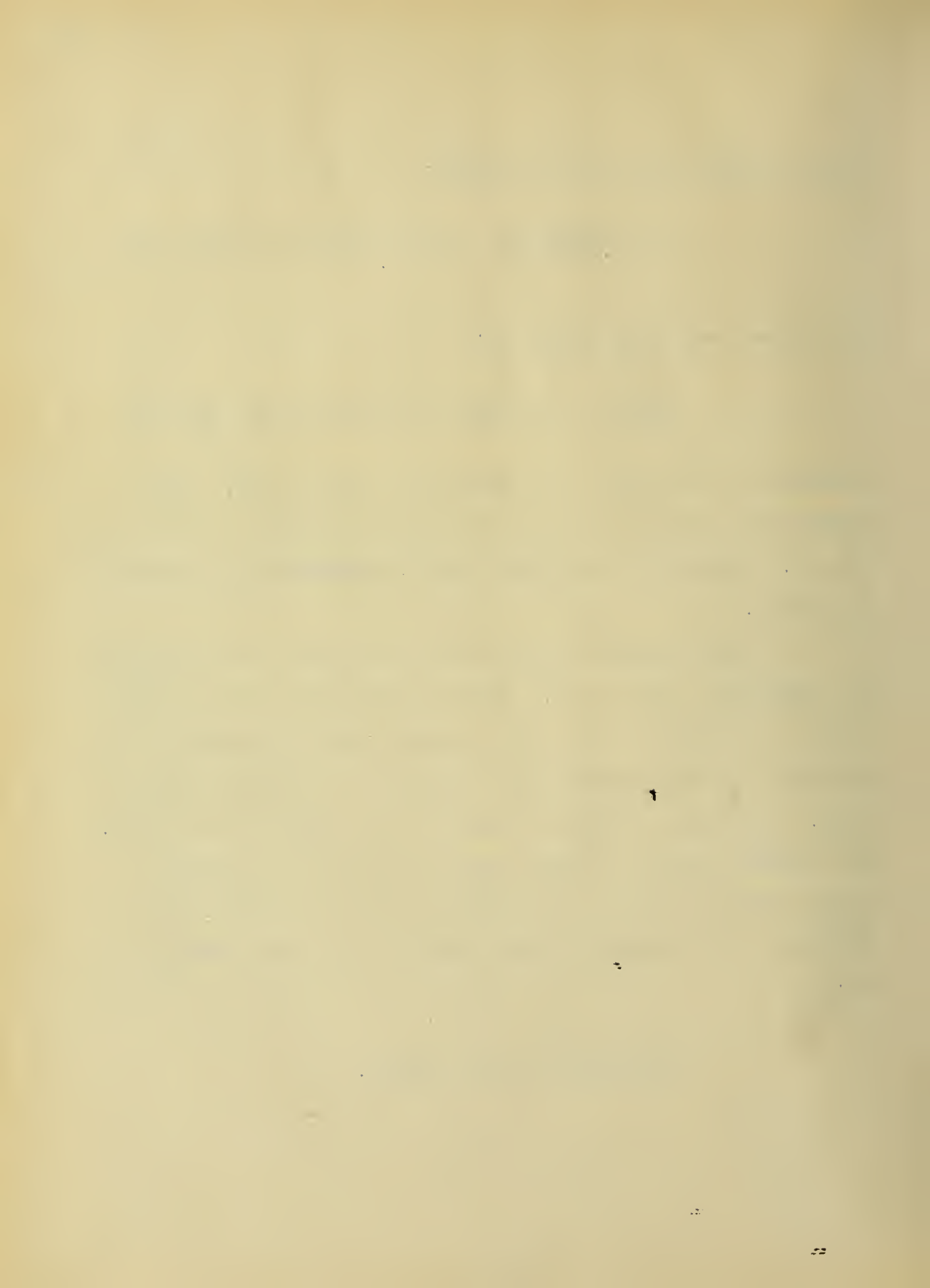
The actual unit bond stress is:

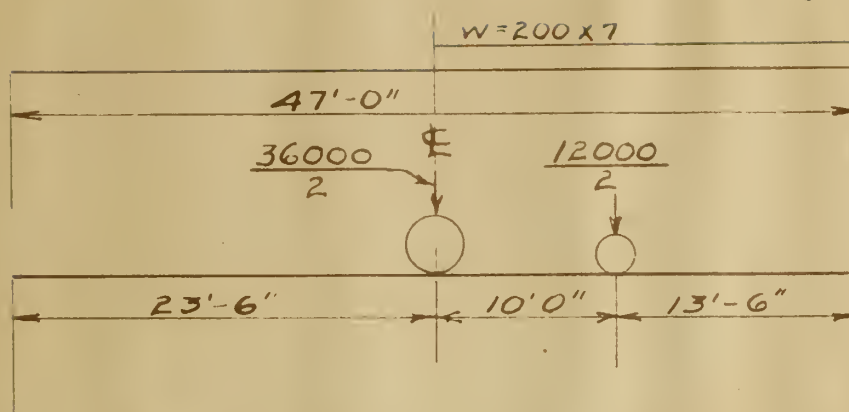
$$\frac{2130}{4 \times 7} = 76 \text{ pounds per square inch, and the}$$

allowable bond stress is 75 pounds per square inch. This deficiency will be overcome by the use of angles as shown in Fig. 3. The bent up rods will not be considered as carrying any shear.

The allowable shear which the concrete will take is 30 pounds per square inch. The stirrups must carry all the stress in excess of this. The maximum shearing stress at the beam end has been computed and equals 100 pounds per square inch. The dead load shear is zero at the center of the beam. The maximum shearing stress over the center section will result from one of the two loadings shown in the Fig. 12. The shear at the center of the beam for the first loading is:

$$\frac{1,400 \times 47}{2 \times 4} = 8,220 \text{ pounds.}$$





*Fig. 12. Loadings for Maximum Center Shear.*

The shear with the roller placed as shown in the second diagram is:

$$\frac{18,000}{2} + 6,000 \times \frac{13.5}{47} = 10,722 \text{ pounds.}$$

The roller produces the greatest unit shear at the center which is:

$$\frac{10,722}{18 \times 40} = 15 \text{ pounds per square inch.}$$

The maximum unit shear diagrams are shown in Plate II. The stress which the concrete will carry is deducted graphically from the total unit stress.

The spacing of the rods is given by the formula:

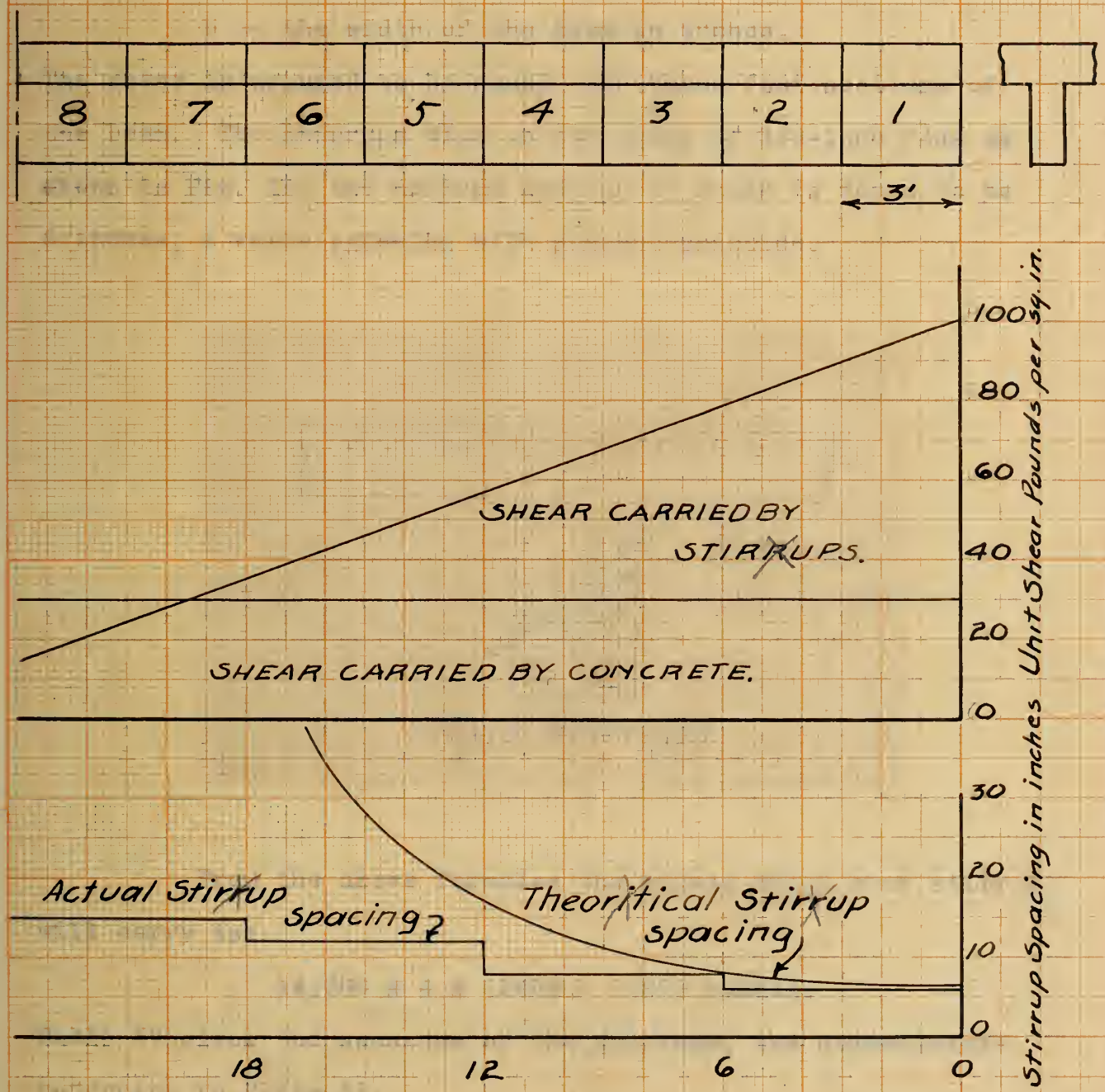
$$S = \frac{P}{vb}$$

where  $s$  = the spacing of the stirrups in inches,





PLATE II

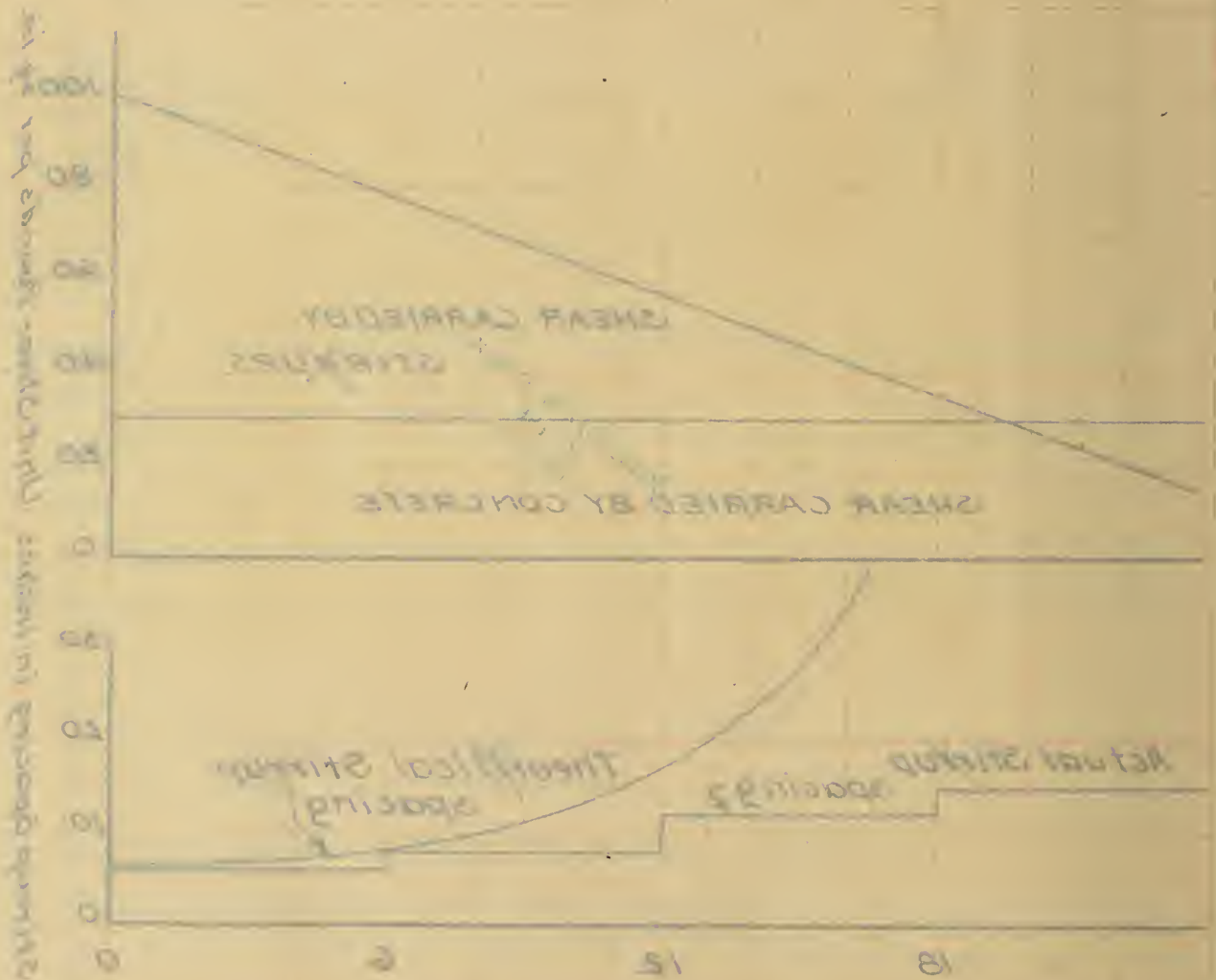


Shear Diagram And Stirrup Spacing For All Floor Girders.

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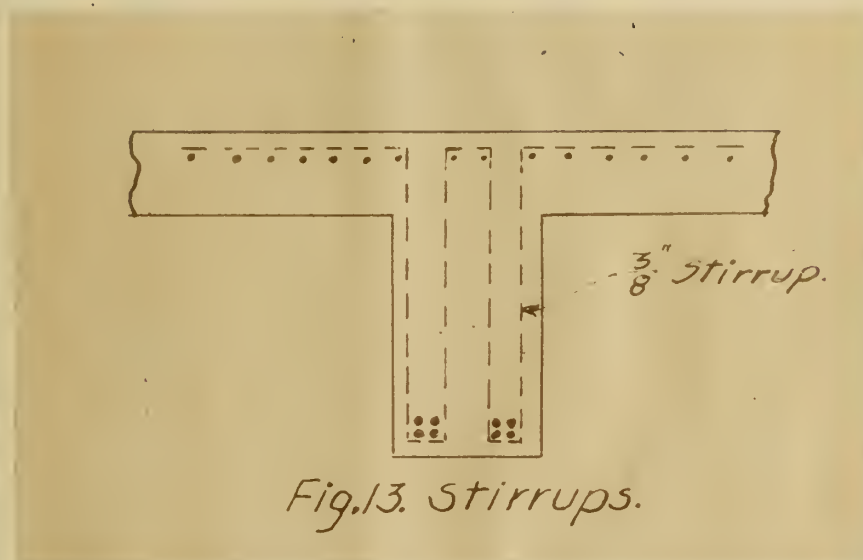


Shear Diagram And Stirrup Spacing for All Floor Girders



$P$  = the stress which each stirrup will carry,  
 $v$  = the unit shear in pounds per square inch, and  
 $b$  = the width of the beam in inches.

The shear is assumed to be equal over three foot sections of the beam. The stirrups will be two loops of 3/8-inch rods as shown in Fig. 13; the minimum spacing of which is found to be 6 inches, a value agreeing with present practice.



From the above formulae the stress which each stirrup will carry is:

$$14,000 \times 4 \times .1406 = 7,875 \text{ pounds.}$$

Table IV gives the spacings of the stirrups, the nomenclature referring to Plate II.

The stirrups will be spaced as follows:

1st: 6 feet, 6 inches

2nd: 6 feet, 8 inches,

3d: 6 feet, 12 inches, and then on 15-inch



centers out to the center of the beam. This spacing is shown graphically on Plate II.

Table IV.  
STIRRUP SPACINGS

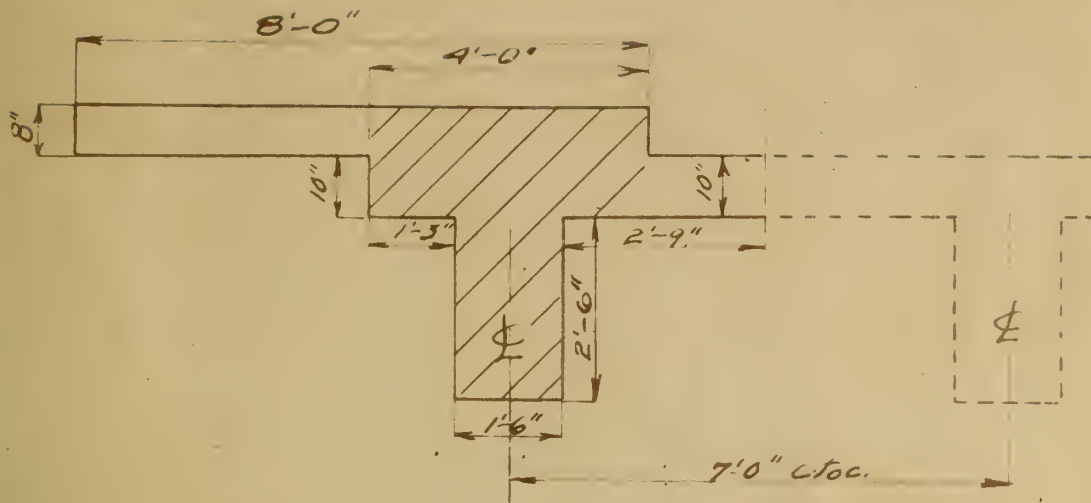
Section	P	v	b	vb	$s \frac{P}{vb}$
1	7875	70	18	1260	6.24
2	7875	58	18	1043	7.50
3	7875	48	18	865	9.10
4	7875	37.5	18	674	11.70
5	7875	27.	18	486	16.20
6	7875	16	18	288	27.40
7	7875	5	18	90	82.5
8	7875	0	18		

The design for the girders under the walks will now be made. The cross section will vary from that of the central girders, See Fig. 1. The live loading for the walk is 100 pounds per square foot. The weight of the railing will be taken as part of the live load as it covers a portion of the walk surface. Fig. 14 shows the section, the shaded area being the tee-beam proper.

Proceeding as in the design of the other Tee-beam, the area of the section is found to be 2,004 square inches 13.91 square feet, and this is equivalent to a uniform weight per lineal foot of:

$$13.91 \times 150 = 2,086 \text{ pounds.}$$





*Fig. 14. Section Through Side Walk And Girder.*

The total weight per foot is:

Dead load	= 2,086 pounds
Live load on walk 8 x 100	= 800 pounds
Live load on roadway 1 1/2 x 200	= <u>300 pounds</u>
Total load	= 3,186 pounds.

The maximum moment is:

$$\frac{3,186 \times 47 \times 47 \times 12}{8} = 10,550,000 \text{ pound-inches.}$$

Here the value of  $j$  will be assumed to be 0.81, the true value of  $j$  being found later. The steel area required to carry the moment is:

$$A = \frac{10,550,000}{0.81 \times 48 \times 14,000}$$

= 19.4 square inches; and the





percent of reinforcement is:

$$\frac{19.4}{48 \times 48} = .842$$

Solving for the true value of  $j$  there results:

$$j = \frac{6 - 6\left(\frac{18}{48}\right) + 2\left(\frac{18}{48}\right)^2 - \left(\frac{18}{48}\right)^3}{6 - 3\left(\frac{18}{48}\right)} \div 2 \times 15 \times .00842$$

$$= 0.816$$

Since the assumed value of  $j$  checks very closely with the true value, no recomputation is necessary.

Before solving for the unit pressure upon the concrete, the value of  $K$  for this beam must be found. It is:

$$K = \frac{P_n + \frac{1}{2} \left(\frac{t}{d}\right)^2}{P_n + \frac{t}{d}}$$

$$= \frac{15 \times .00842 + \frac{1}{2} \left(\frac{18}{48}\right)^2}{15 \times .00842 + \left(\frac{18}{48}\right)}$$

$$= 0.396$$

and using this the value of  $F_c$  is determined as follows:

$$10,550,000 = F_c \left(1 - \frac{18}{2 \times 0.396 \times 48}\right) 48 \times 18 \times 0.816 \times 48.$$

$$F_c = 570 \text{ pounds per square inch.}$$

The maximum end shear is produced when the walk is entirely loaded and equals:

$$3,186 \times \frac{47}{2} = 75,250 \text{ pounds, from which the}$$



unit shear is computed to be,

$$\frac{75,250}{18 \times 48} = 87.5 \text{ pounds per square inch.}$$

The maximum shear at the center will be produced when the live load covers one-half of the span and equals it is:

$$\frac{800.0 \times 47}{8} = 4700.0 \text{ pounds, from which the unit}$$

shear at the center is:

$$\frac{4700.0}{18 \times 48} = 5.43 \text{ pounds per square inch.}$$

These unit shears just computed are less than the unit shears for the center beams. The spacings of the stirrups will be the same in the girders at the sides of the structure as those in the center.

The steel area of 19.4 square inches necessary to carry the maximum moment will be made up of the following rods.

$$6 - 1 \frac{1}{2}\text{-inch rods} = 6 \times 2.25 = 13.50 \text{ square inches.}$$

$$2 - 1 \frac{3}{4}\text{-inch rods} = 2 \times 3.06 = \underline{6.12 \text{ square inches.}}$$

$$\text{Total area} = 19.62 \text{ square inches.}$$

Two of the 1 1/2-inch rods and two of the 1 3/4-inch rods will be carried directly through the girder. The other four 1 1/2-inch rods will be turned up in pairs. The length which the first pair must have is:

$$x = \sqrt{\frac{47}{19.62}} \sqrt{4.5}$$

$$= 22.3 \text{ feet.}$$

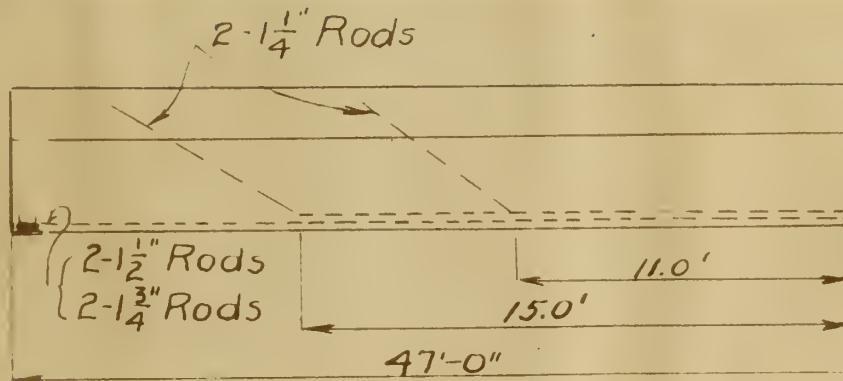


This pair of rods may be turned up at a distance of 11 feet from the center. The second pair of rods will be carried to the following length:

$$x = \sqrt{\frac{47}{19.62}} \sqrt{9.0}$$

$$= 30.6 \text{ feet.}$$

This pair of rods will be turned up at a distance of 15 feet from the center of the beam. Fig. 15 shows the spacing.



*Fig. 15 Position of Bent Up Rods in Side Girders.*

The bond stress necessary at the end is:

$$U = \frac{75,250}{0.81 \times 48} = 1,940 \text{ pounds per lineal inch}$$

of rods; and the unit bond stress is:

$$\frac{1,940}{2 \times 6} \frac{1}{2 \times 7} = 74 \text{ pounds per square inch. Angles}$$

will be used in addition as shown in Fig. 3.

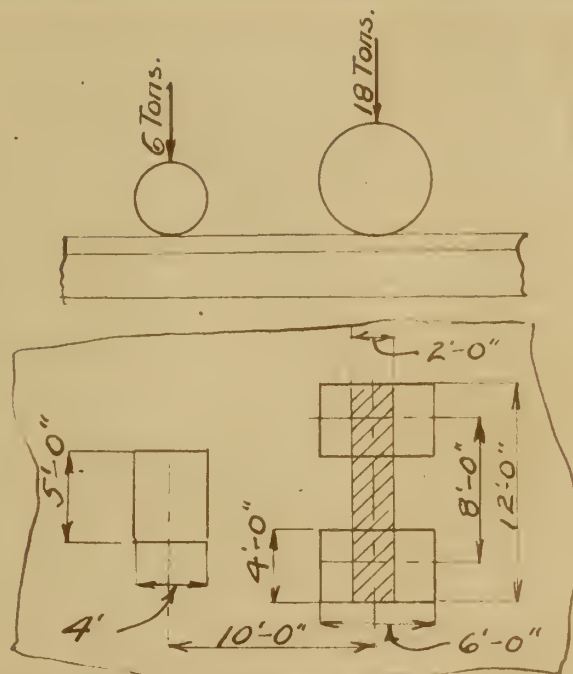
The slab rods running perpendicular to the girders will now be designed. The loads will be assumed to be transmitted laterally to the girders and then longitudinally





by the girders to the piers. The slab rods running parallel to the girders will be designed later.

The rear wheels of the roller will give the largest loading which the cross rods must be designed to carry. Because the slab is reinforced in two directions, it will be assumed that the load of 18 tons concentrated upon the rear axle will be uniformly distributed over an area 12 by 2 feet. See Fig. 16.



*Fig. 16. Equivalent Roller Loading.*

The equivalent load per lineal foot of a two-foot portion is:

$$\frac{18 \times 2,000}{12} = 3,000 \text{ pounds, and the dead weight}$$

of this same portion is:



$$\frac{10 \times 24 \times 150}{144} = 250 \text{ pounds per lineal foot.}$$

The slab is continuous over the girders. Considering the length, 7 feet, as the span of this element of slab as shown in Fig. 16, the moment, neglecting the supporting influence of that portion of the floor adjoining it, is:

$$M = \frac{(3,000 + 250) \times 7 \times 7 \times 12}{10}$$

$$= 159,200 \text{ pound-inches.}$$

The coefficient of  $1/10$  is used rather than  $1/8$  because of the continuity of the floor over the girders. The above equivalent loading has been approximately made, and it is consistent to finish the computation using the diagrams in the text.

The value of  $R$  is:

$$R = \frac{M}{bd^2}$$

where

$R$  = the modulus of rupture,

$M$  = the total moment,

$b$  = the width of the slab, and

$d$  = the effective depth.

Considering  $d$  as 1 inch less than the depth of the slab and using the maximum moment as just computed:

$$R = \frac{159,000}{24 \times 9 \times 9}$$

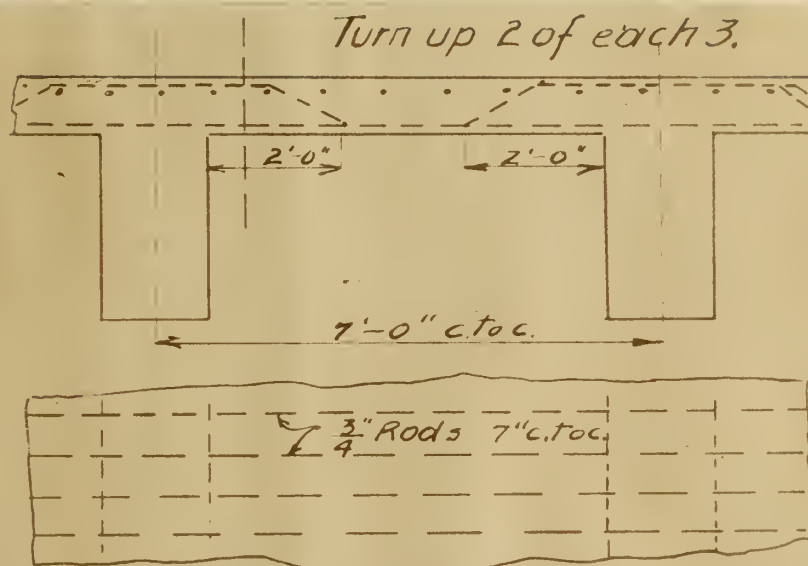
$$= 81.7$$

Entering the diagram with this value of  $R$ , and using a percentage of reinforcement of 0.8 percent, the unit stresses in the



steel and concrete are found to be 12,000 and 480 pounds per square inch respectively. These stresses are lower than the allowable values of 14,000 and 600 pounds, but as some excess should be allowed to provide for temperature stresses, a re-computation will not be made. The percent of 0.8 can be supplied by 3/4-inch rods on 7-inch centers.

In order to care for the negative moments over the girders, two-thirds of these rods will be turned up over the girders, near the surface of the slab. See Fig. 17.



*Fig. 17. Slab Cross Reinforcement.*

The value of the negative moment over the supports may be taken as  $1/10 w l^2$  (p. 303) if the width of the support is neglected, but since 18 inches, the width of the supporting girder, is less than twelve times the span length, in this case 7 feet, the spacing of the girders, the moment over the edge of the support is reduced about 25 percent of the value of the moment over the theoretical center of support. Furthermore, the above moment is computed for localized loading and





as the slab is greatly strengthened by the adjoining structure, it would seem from this and the above reasons that the assumed reduction of 25% of the value of theoretical moment is a safe value for use in computation.

The theoretical value of the moment over the support is equal to the assumed value already computed as the moment at the center of the slab span, and is 159,000 pound-inches. 75 percent of this, the moment used in computation, is:

$$\begin{aligned} -M &= 0.75 \times 159,000 \\ &= 119,500 \text{ pound-inches.} \end{aligned}$$

The modulus of rupture is:

$$\begin{aligned} R &= \frac{119,500}{24 \times 9 \times 9} \\ &= 61.5 \end{aligned}$$

Since two out of every three rods are turned up over supports, the steel area taking negative moment is  $2/3$  of that taking positive moment, or:

$$\begin{aligned} p &= 0.66 \times 0.8 \\ &= 0.528 \text{ percent.} \end{aligned}$$

Entering the table with this value of the reinforcement and the above value of R, the stresses in the concrete and steel are found to be 425 and 13,500 pounds per square inch respectively, which are less than the allowable stresses of 600 and 14,000 pounds per square inch.

The shear at the edge of the girder, see sect. a-a<sub>1</sub> Fig. 17, is:



$$\frac{3,000 \times (3\frac{1}{2} - \frac{1}{2} \times 1\frac{1}{2})}{10 \times 24} = 34 \text{ pounds per square}$$

inch, which is less than the allowable which is 100 pounds per square inch.

The rods in the floor slab running parallel to the girders are chosen arbitrarily as 3/4 inches in diameter, spaced on one-foot centers. The percent of reinforcement in this case is:

$$p = \frac{0.5625}{12 \times 10} = 0.69$$

These rods will aid in distributing any concentrated load and also tend to secure greater rigidity of the floor system.

## VI DESIGN OF SIDEWALKS.

The side-walk slab is a short cantilever projecting out from the side girder. Alternate rods in the floor slab will be continued through the side girder and out into the walk. See Fig. 18. The rods are spaced on 7-inch centers in the main slab. This will provide one 3/4-inch rod for every 14 inches of walk. The moment which this rod must carry is:

$$M = \left( 100 \times 1.16 + \frac{8 \times 14 \times 150}{144} \right) \times 4 \times \frac{4}{2} \times 12$$

$$= 22,400 \text{ pound-inches.}$$

Proceeding as before the section modulus is:

$$R = \frac{M}{bd^2}$$

$$= \frac{22,400}{14 \times 7 \times 7} = 32.6, \text{ and the percent of}$$



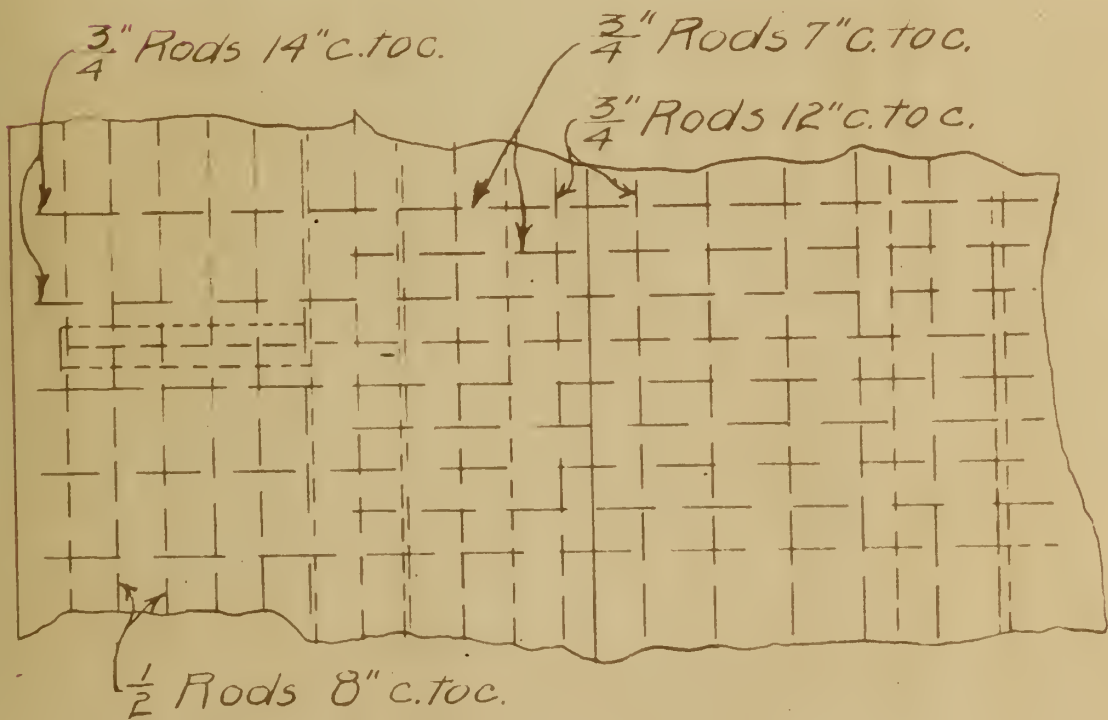
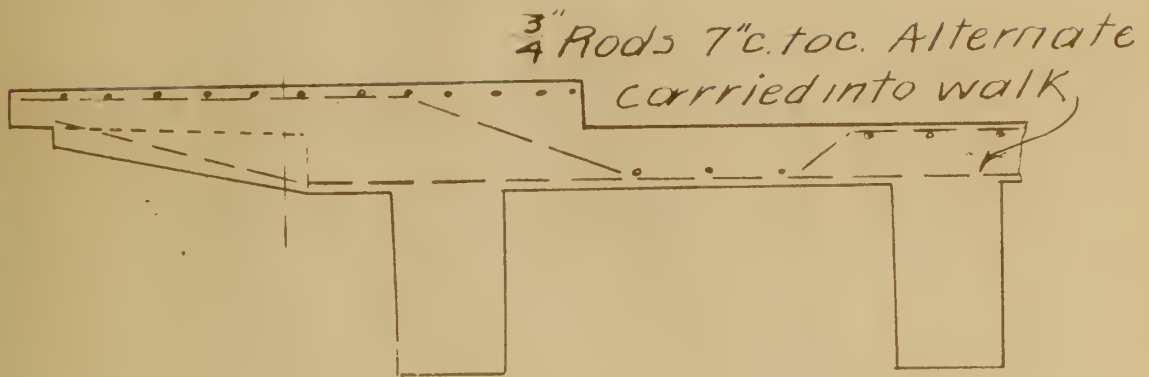


Fig. 18. Cantilever Reinforcement in Walk





reinforcement is:

$$\frac{0.5625}{14 \times 10} = 0.40$$

From the diagram on p. 277 with R 31.7 and the percent of reinforcement equal to 0.4, the stress in the concrete is found to be 280 pounds per square inch and that in the steel to be 10,000 pounds per square inch, which are less than the allowable stress. The cantilever as designed is of a uniform thickness of 8 inches. It will be made 6 inches thick at the outer edge, as at that point the moment is zero and a reduction in the thickness of the slab will tend to decrease the dead load upon the cantilever. For artistic appearance, small brackets will be built under the ledge. They will not be designed to carry any stress; however, one of the floor rods will be continued out through the side girder as shown in Fig. 18, in this way strengthening the danger section a-a.

The longitudinal walk rods will be 1/2-inch in diameter, spaced on 8-inch centers.

To further the symmetrical appearance of the structure, the main railing posts will be built at the edge of each walk over the piers. The intermediate posts will divide the span length of 47 feet into four equal panel lengths of 11 feet 9 inches. This will give an unsupported length of 10 feet to the rail slab proper. Fig. 19 shows a section and elevation of the rail slab and posts. The recesses in the post being coated with asphalt will prevent a rigid union between the two members.



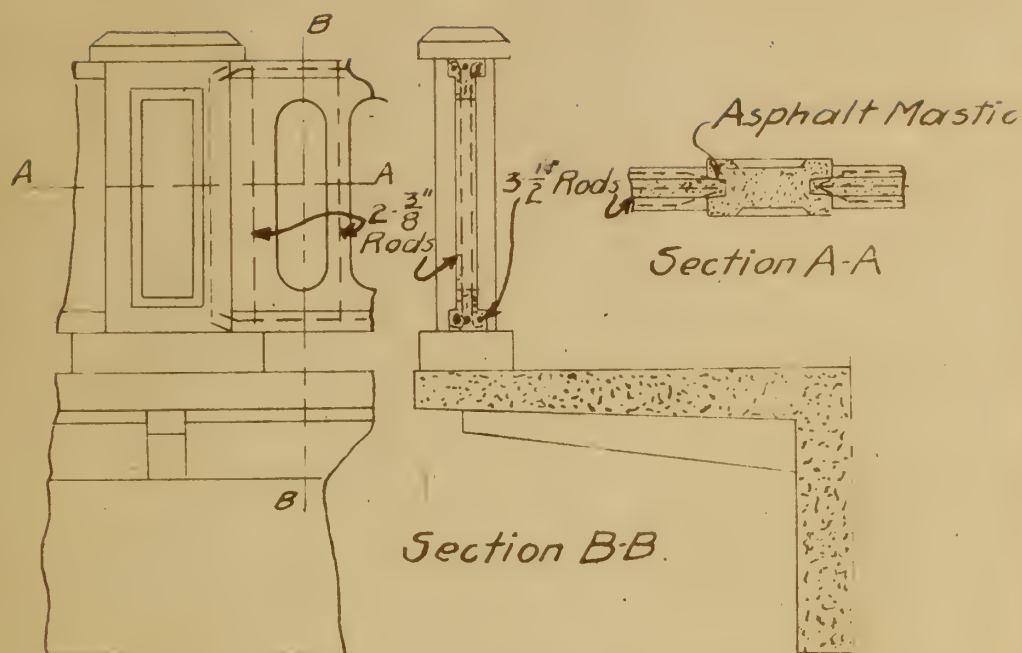


Fig.19. Reinforcement of Railing And Expansion Joint.

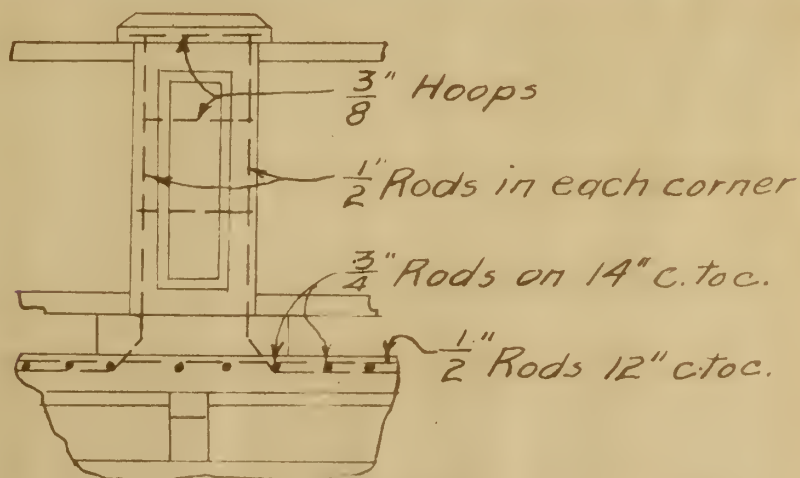
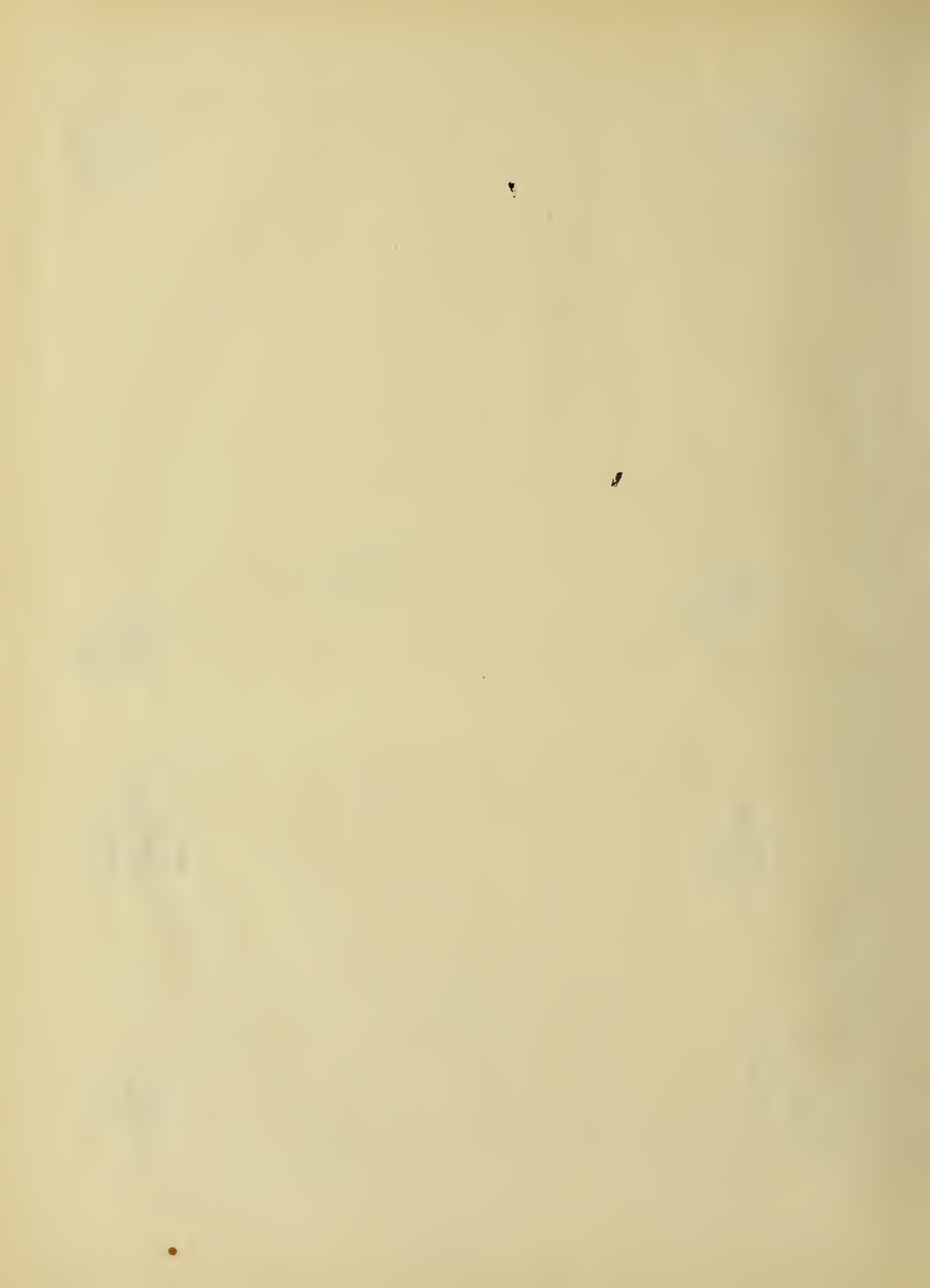


Fig.20. Intermediate Post Reinforcement.



The weight of the rail slab per foot is computed to be 87 pounds, and the moment at the center is:

$$M = \frac{87 \times 10 \times 10 \times 12}{8}$$

$$= 13,050 \text{ pound-inches.}$$

The steel in each flange of the rail will be designed to carry the dead load. The distance center to center of the rods is:

42-1 1/2-1 1/2 39 inches, and the area of steel necessary, then, to carry the moment from dead load is:

$$A = \frac{13,050}{39 \times 14,000}$$

$$= 0.024 \text{ square inches.}$$

This could be supplied by 1 - 1/4 inch rod but the reinforcement will be increased to 3 - 1/2-inch rods or an area of:

$$3 \times .25 = 0.75 \text{ square inches.}$$

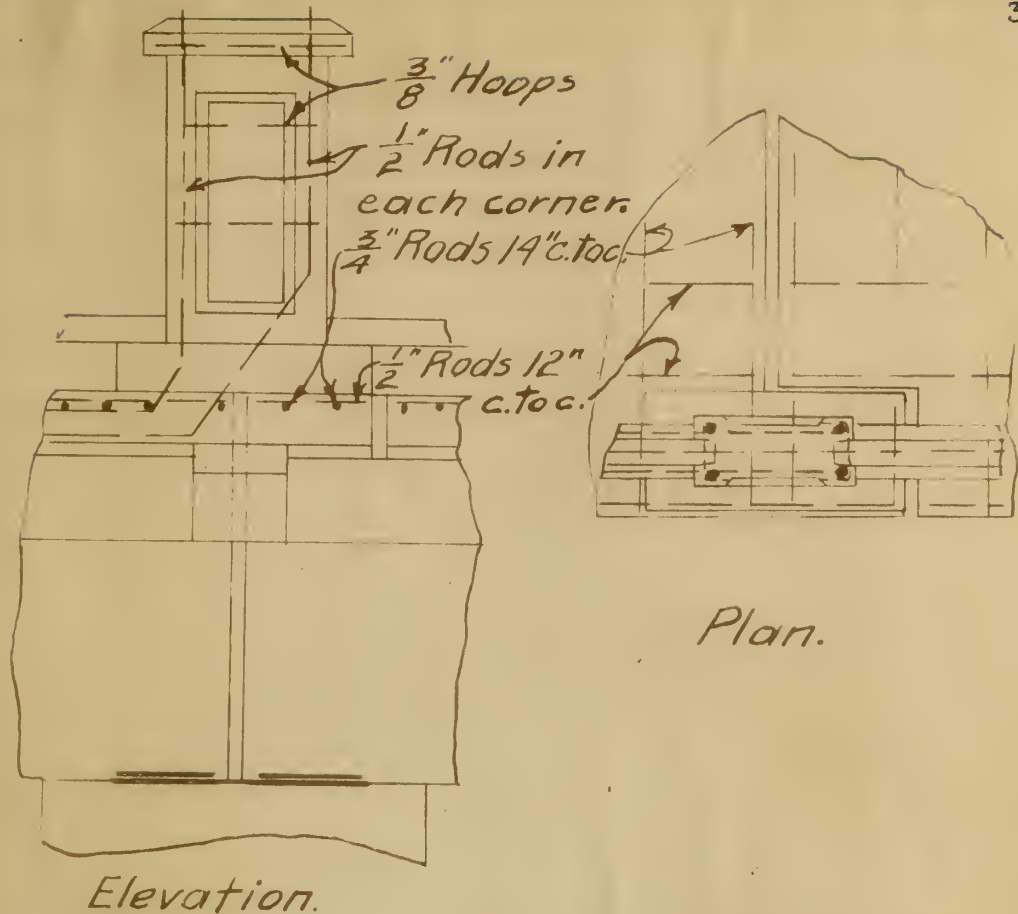
The web reinforcement will consist of 2 - 3/8-inch rods run up through each baluster, looping the flange rods at both the top and bottom.

The intermediate posts will be tied into the floor slab by short lengths of 1/2-inch rods. The entire reinforcement of the intermediate posts is shown in Fig. 20. The size and position of this reinforcement is determined by practice rather than by any computations.

Because of the expansion joint in the floor system over the piers, the base of the railing post at this point must be made of special design. Fig. 21 shows the elevation







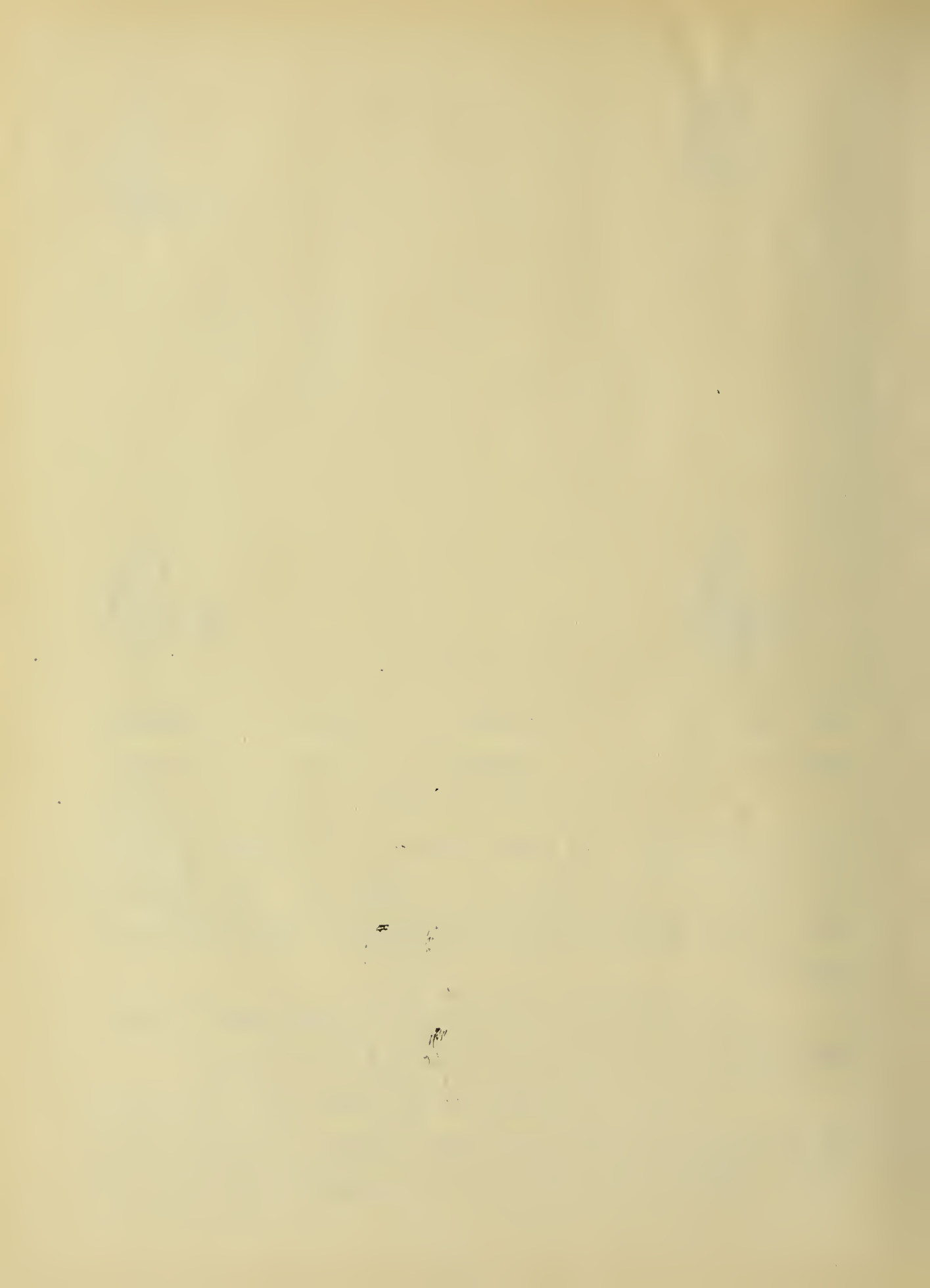
*Fig. 21. Reinforcement of Pier Rail Posts.*

and plan of the pier rail-posts. The width of the brackets under the walk will be increased to 12 inches at this point.

Four  $\frac{3}{4}$ -inch bolts 12 inches long will be imbedded in the top of the pier posts, leaving 1  $\frac{1}{2}$  inches of the thread end projecting above the top of the post cap. This will provide an effective means of rigidly bolting the light post to the railing.

No design for the light post proper will be made, as many different patterns can be secured on the market.

To carry the wires for the lights from the post to the wire cross-arm below the floor system, a  $\frac{3}{4}$ -inch conduit will be sufficient. This will be imbedded in the floor slab,



starting at the main feed wires and ending in the top of the rail posts which will carry the lights, see Fig. 2.

## V DESIGN OF PIERS.

The type of pier has been indicated in Fig. 2. The computations do not lend themselves to as rigid an analysis as those of the floor system, and certain assumptions must be made which if incorrect will be on the side of safety.

The dead load which each pier will carry is the weight of 57 feet of floor system. The greatest live load which will be concentrated upon one pier will result from either the uniform loadings of 200 pounds per square foot upon the roadway and 100 pounds per square foot upon the walk, or from the roller when directly over the pier. The total live and dead load per lineal foot of central girders has already been computed and equals 3,225 pounds. The maximum load upon one girder seat is then,

$$3,225 \times 47 = 151,600 \text{ pounds.}$$

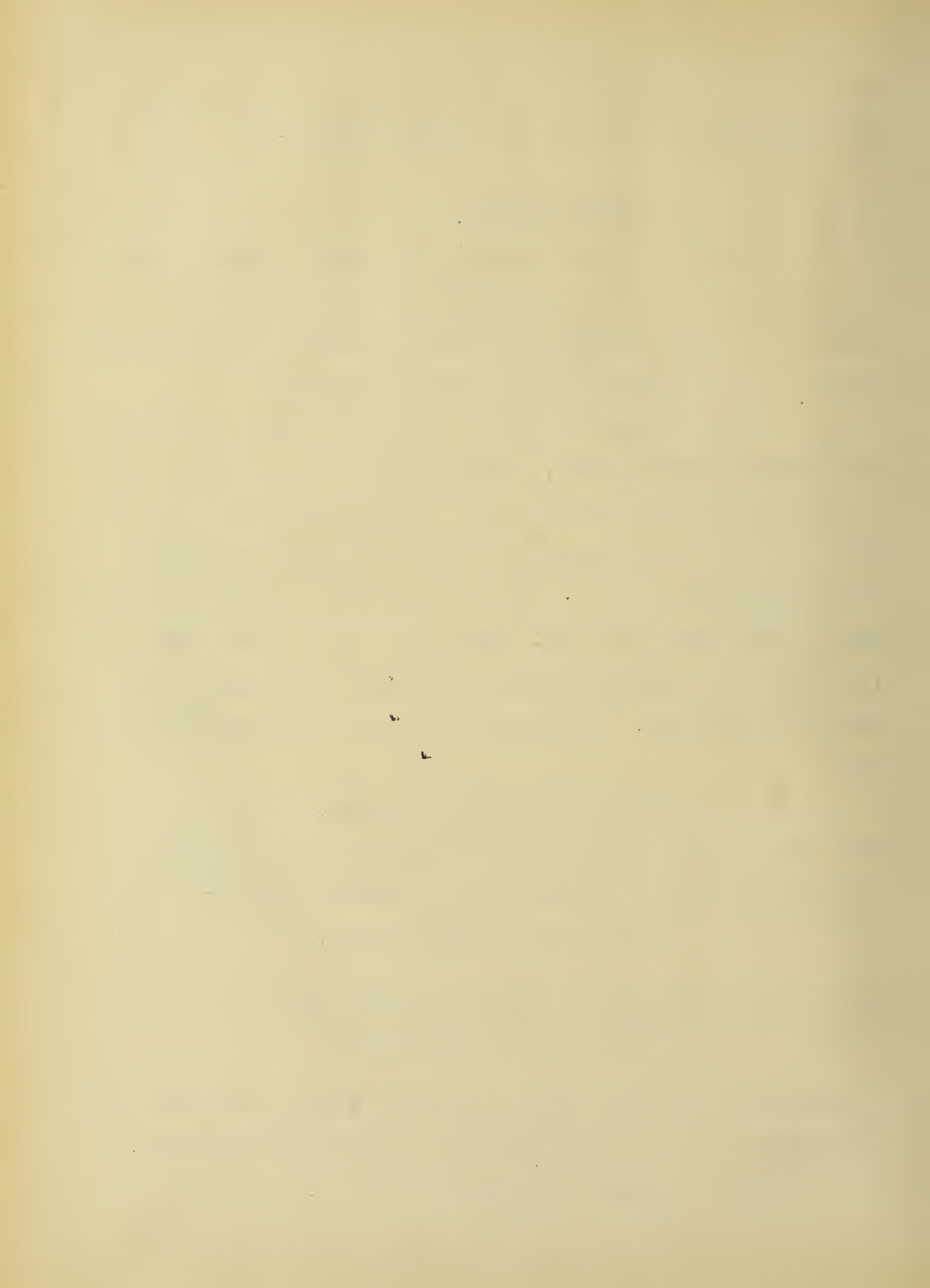
The total load per lineal foot for uniform loading upon the side girders is 3,186 pounds, and the maximum load is:

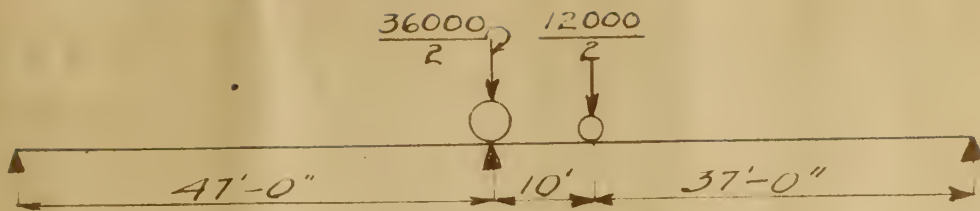
$$3,186 \times 47 = 149,800 \text{ pounds.}$$

The dead load reaction at the center girder seats is:

$$1,825 \times 47 = 85,700 \text{ pounds.}$$

In computing the roller load reactions, it will be assumed that the weight of the roller is carried by two adjacent girders. The roller will be placed as shown in Fig. 22.





*Fig. 22. Position of Roller Giving Maximum Reaction*

The load which comes upon each of the two longitudinal seats from the roller placed as above is:

$$L = \frac{36,000}{2} + \frac{12,000}{2} \times \frac{37}{47}.$$

$$= 22,720$$

The total load concentrated is:

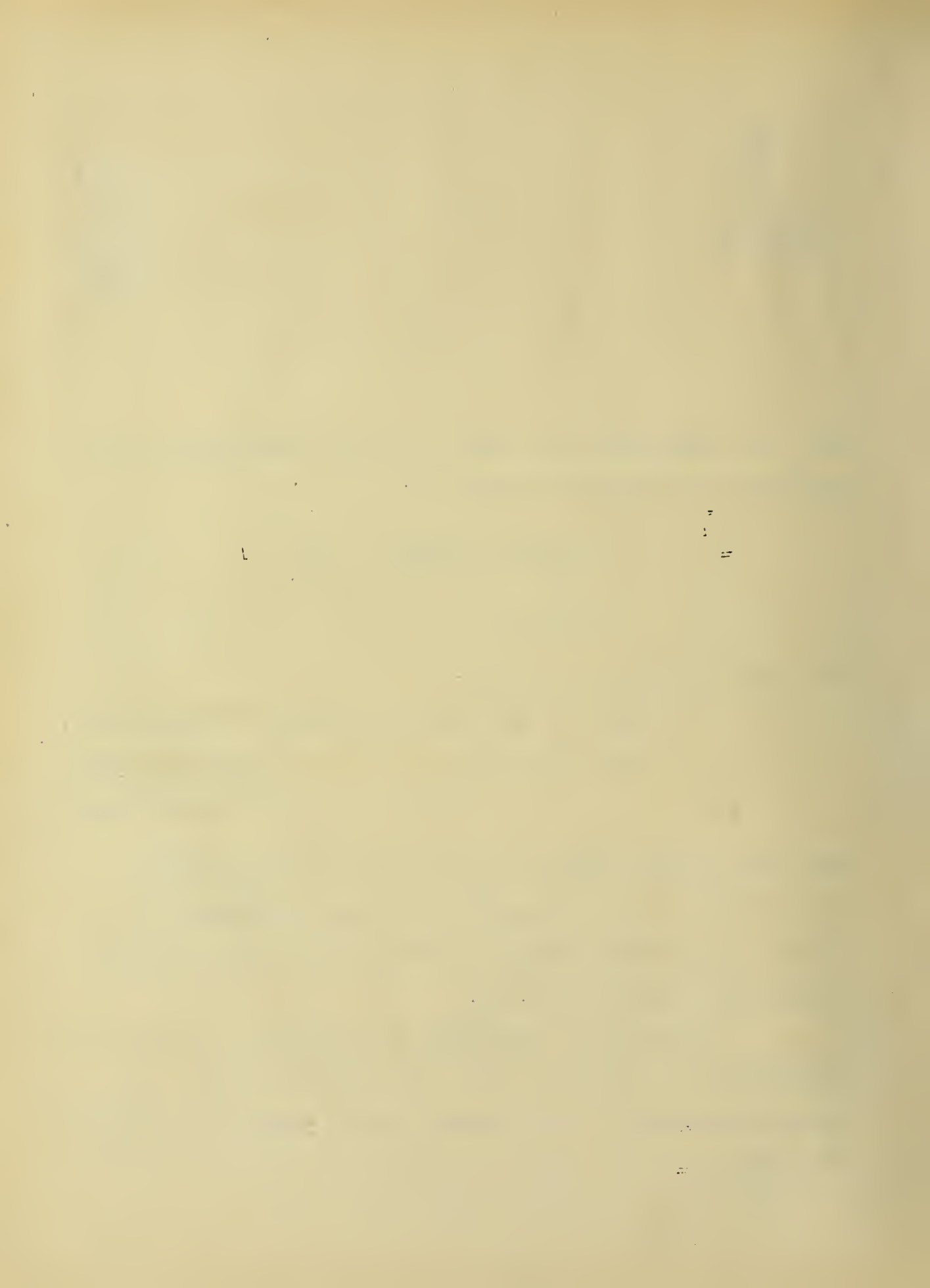
Weight of 47 feet floor system = 85,700 pounds.

Weight of roller = 22,720 pounds

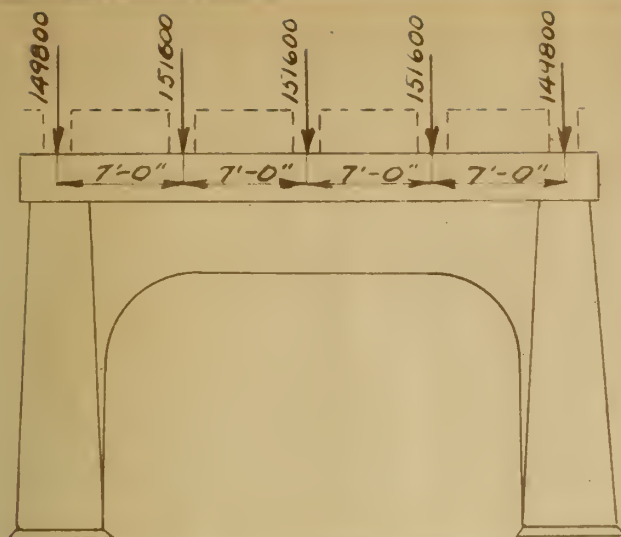
Total = 108,420 pounds.

The total load as computed from uniform and dead load is 151,600 pounds, which is greater than that produced by the roller. The loads for which the beam of the pier must be designed as shown in Fig. 23.

The method of computing the stresses and determining reinforcement is identical with that used in designing the floor beam girders. The stresses will be taken from the diagram on p. 283.





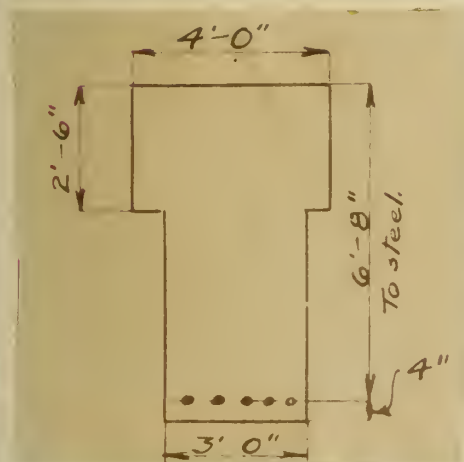


*Fig. 23. Maximum Loads on Pier.*

The maximum moment at the center of the beam due to the floor system and loads is:

$$\begin{aligned}
 M &= [151,600 \times 14 - 151,600 \times 7] \times 12 \\
 &= 25,500,000 \text{ pound-inches.}
 \end{aligned}$$

The section of the beam of the pier is shown in Fig. 24.



*Fig 24. Cross Section of Cross Beam of Pier.*

The massive web is necessary to care for the shear. This will be shown later. The dead weight of the beam is computed to be 3,523 pounds per lineal foot. The moment, caused by this load is:

$$M. = 3,523 \times 28 \times \frac{28 \times 12}{8}$$

= 4,150,000 pound-inches, at the center and



the total moment is:

$$\text{Dead load} = 4,150,000 \text{ pound-inches.}$$

$$\text{Floor system} = \underline{25,500,000 \text{ pound-inches.}}$$

$$\text{Total} = 29,650,000 \text{ pound-inches.}$$

From which the required section modulus is determined to be:

$$R = \frac{29,650,000}{48 \times 80 \times 80}$$

$$96.5$$

and the ratio of the flange thickness to the effective depth is:

$$\frac{t}{d} = \frac{30}{74}$$

$$= 0.40$$

Entering the table with these values of  $R$  and  $\frac{t}{d}$ , the unit pressure on the concrete is found to be 580 pounds per square inch, and the corresponding value of  $j$  is 0.88. The steel area required is:

$$A = \frac{29,650,000}{0.88 \times 80 \times 14,000}$$

$$= 30.1 \text{ square inches which can be made}$$

up of 16 - 1 3/8-inch rods.

The maximum shear at the end of the beam is:

$$V = 151,600 \times 1 \frac{1}{2} + 3,523 \times 14$$

$$= 276,700 \text{ pounds}$$

from which the unit shear is:

$$u = \frac{276,700}{36 \times 80}$$

$$= 96 \text{ pounds per square inch.}$$



The necessary bond strength which must be developed per lineal inch at the ends of the rods which are laid continuous through the bottom of the girder is:

$$U = \frac{276,700}{0.88 \times 80}$$

$$= 3,930 \text{ pounds.}$$

Allowing 6 of the 1 3/8-inch rods to run directly through, the bond stress per inch which must be developed to carry the above is:

$$U = \frac{3,930}{6 \times (4 \times 1 \frac{3}{8})}$$

$$= 119 \text{ pounds per square inch, which is greater}$$

than the allowable bond stress of 100 pounds per square inch.

This deficiency is overcome by hooking the bars around vertical rods in the columns as shown in Fig. 24.

The remaining ten of the 16 rods in the bottom of the girder will be turned up in pairs. The total lengths of these pairs before turning up is computed as follows:

$$X_1 = \sqrt{\frac{28}{30.1}} \sqrt{2 \times 1.89}$$

$$= 10.7 \text{ feet.}$$

$$X_2 = \sqrt{\frac{28}{30.1}} \sqrt{4 \times 1.89}$$

$$= 15.1 \text{ feet.}$$

$$X_3 = \sqrt{\frac{28}{30.1}} \sqrt{6 \times 1.89}$$

$$= 18.5 \text{ feet.}$$

$$X_4 = \sqrt{\frac{28}{30.1}} \sqrt{8 \times 1.89}$$

$$= 21.4 \text{ feet.}$$





$$X_5 = \sqrt{\frac{28}{30.1}} \sqrt{10 \times 1.89}$$

$$= 23.8 \text{ feet.}$$

The distances from the center at which they may be turned up is equal to one-half of the above distances or:

5.3 feet.

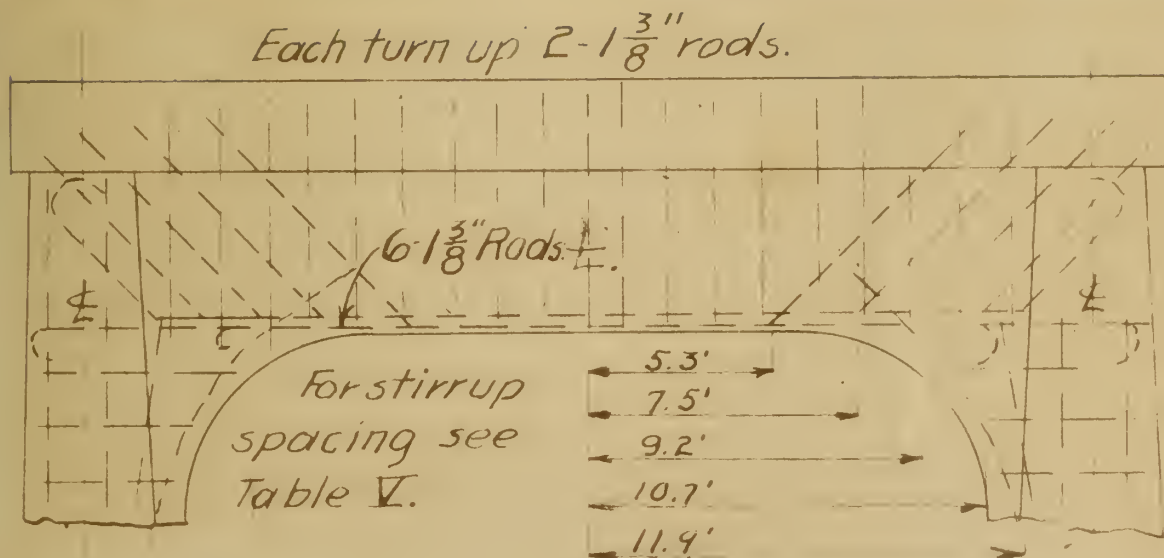
7.5 feet.

9.2 feet.

10.7 feet.

11.9 feet.

Fig. 25 shows the points at which the rods are to be turned up.



*Fig.25. Reinforcement in Pier.*

The shear diagram must now be constructed, in order to determine the spacings of stirrups. These stirrups will



consist of a double loop of 1/2-inch rods. The stress which each stirrup will carry is:

$$P = 4 \times 0.25 \times 14,000$$

$$= 14,000 \text{ pounds.}$$

The shear is assumed to be equal over three foot sections and the turned-up rods of the reinforcement are assumed to carry no shear. The spacings of the stirrups are tabulated in Table V.

Table V.

Theoretic Stirrup Spacings in Pier.

Section (See Plate III)	P	v	b	vb	$s = \frac{P}{vb}$
1	14,000	66	36	2,380	5.9 inches
2	14,000	45	36	1,620	8.65 "
3	14,000	25	36	900	15.60 "
4	14,000	4	36	144	102.00 "

The stirrups will be spaced as follows, starting from the middle of the end columns.

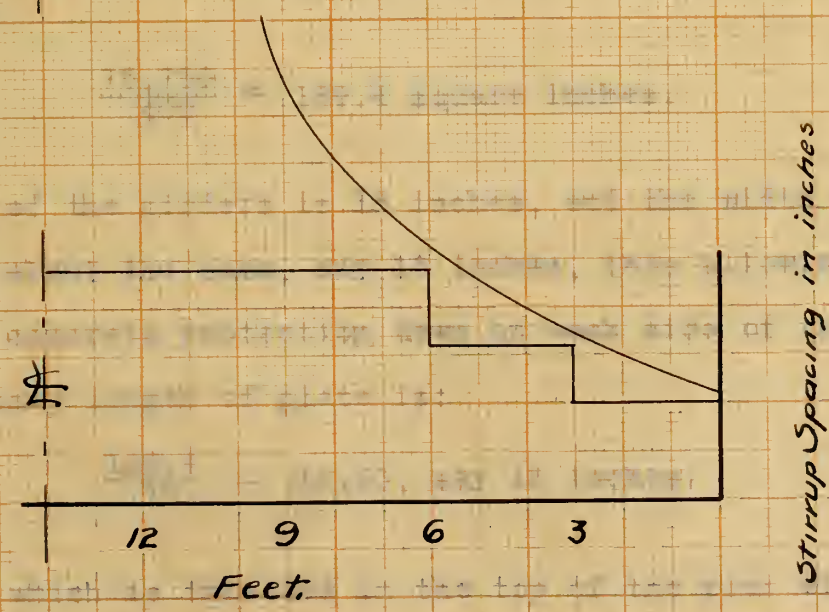
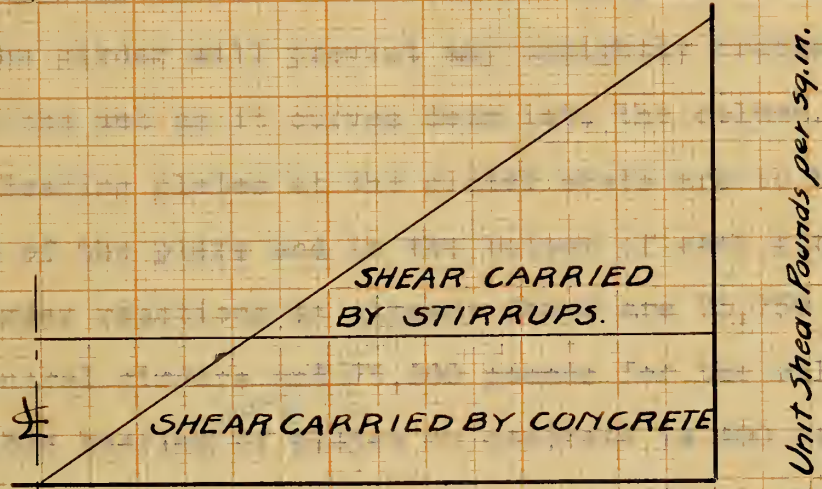
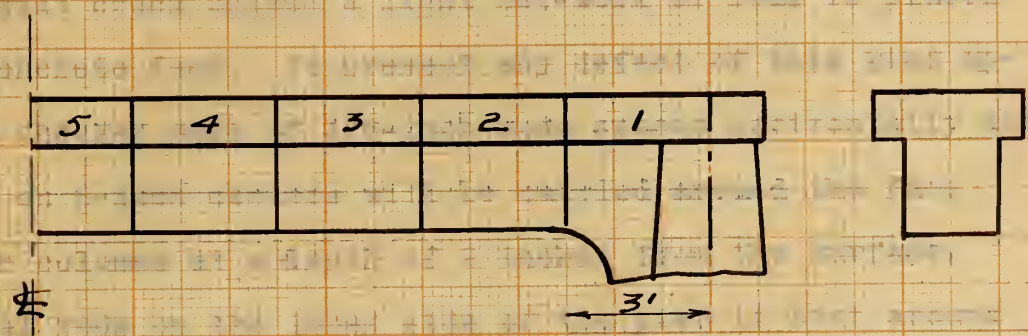
First 3 feet: 5 inches.

Second 3 feet: 8 inches, and then on 12-inch centers out to the center of the span. Plate III shows the shear diagram as well as the theoretical and actual spacing of stirrups.

The columns will not be reinforced for compression since the cross-section is large enough to carry any possible load.



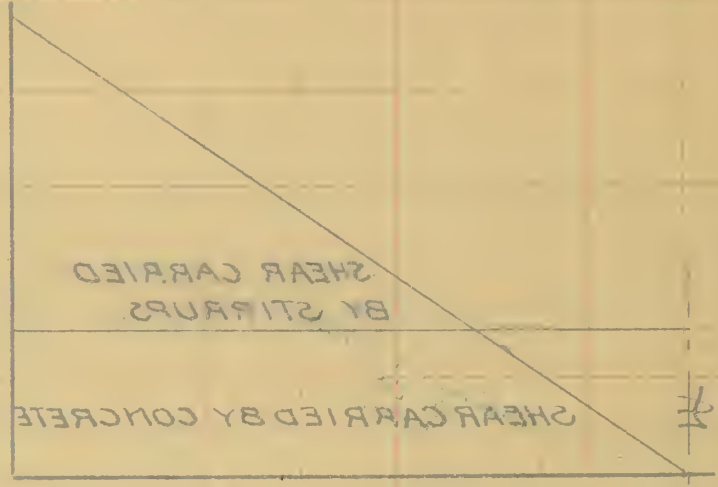




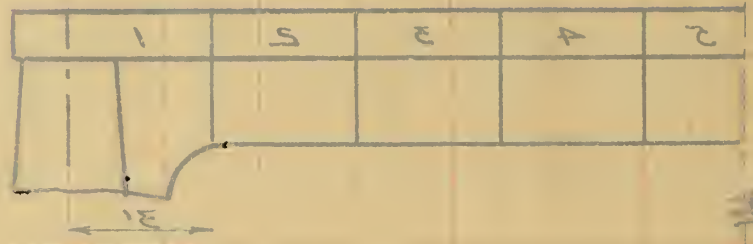
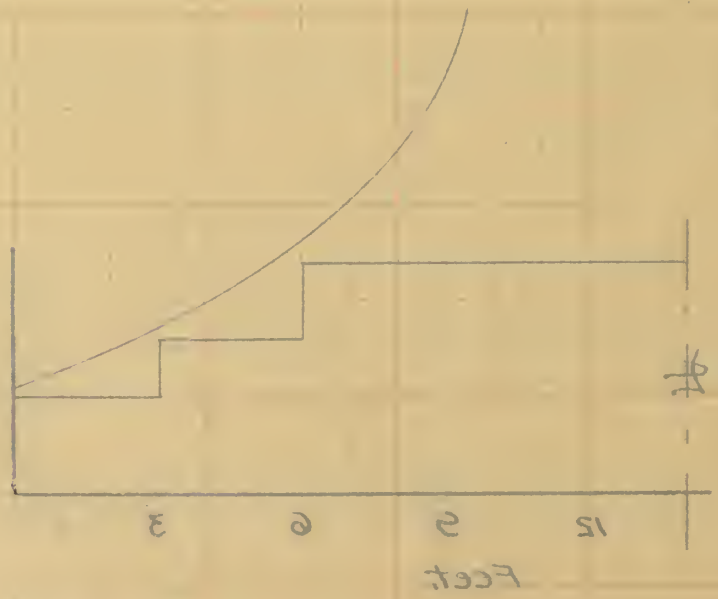
Shear Diagram And Stirrup Spacing  
For Pier Girders.

# Shear Diagram And Stirrup Spacing For Pier Girders.

Shear Carried By Concrete



Stirrup Spacing





Experience has shown that the surface of a massive structure will crack unless a light net-work of rods is placed near the concrete face. To prevent any defect of this kind appearing, a checker work of 3/8-inch rods spaced horizontally and vertically on 8-inch centers will be carried around the face of the pier columns at a depth of 2 inches from the surface. The vertical rods on the inner side of the pier if bent around and into the girder will prevent any unsightly cracks from appearing in the web as it curves down into the column.

Bearing plates at the girder seats are to be imbedded in the top of the piers and in the bottom of each girder. The maximum girder reactions as already found are 75,750 pounds for the central girders and 75,200 pounds for the walk girders. The allowable bearing of plates on concrete is 400 pounds per square inch. The necessary area of bearing for each girder is therefore:

$$\frac{75,750}{400} = 189.4 \text{ square inches.}$$

The width of the girders is 18 inches, and the width of plate should be about the same, say 15 inches, thus allowing 1 1/2 inches of concrete protection down on each side of the plate. The necessary length of plate is:

$$\frac{189.4}{15} = 12.63, \text{ say } 13 \text{ inches.}$$

The plate which is imbedded in the top of the pier will have to be twice the length of the girder bearing plates as above computed and should be at least 6 inches larger in order to allow



for expansion of the girders. The bottom plates will be slotted at one end only. The bolts are taken as 7/8-inch in diameter. As the function of the bolts in the slots is to keep the girders plates in correct relative position, the stress which they take cannot be computed. The eight 7/8" bolts will have to carry in shear the temperature stresses carried from the plates into the reinforcement. Assuming an allowable shearing stress of 9,000 pounds per square inch, the total stress which these bolts can take is:

$$8 \times 9,000 \times .601 = 43,000 \text{ pounds}$$

As will be shown later, this is greater than the probable temperature stress at this point.

Allowance should be made for 150 degrees of temperature, which is sufficient to cover the greatest variation of this climate. The coefficient of expansion for concrete per unit length is 0.000,006 and the amount of expansion for one span for 150 degrees difference in temperature is:

$$0.000,006 \times 47.0 \times 150 = 0.042 \text{ foot.}$$

This is about 1/2 inch. The total length of slot should be  $1/2 + 7/8$ , say 2 inches. The plates in the girder ends will be bolted to the ends of the reinforcing rods as shown in Fig. 26.

The additional tension which will be put upon the reinforcement when any movement takes place between the plates will now be computed. The maximum pressure upon any plate is 75,750 pounds. The coefficient of friction is 0.2 for cast iron



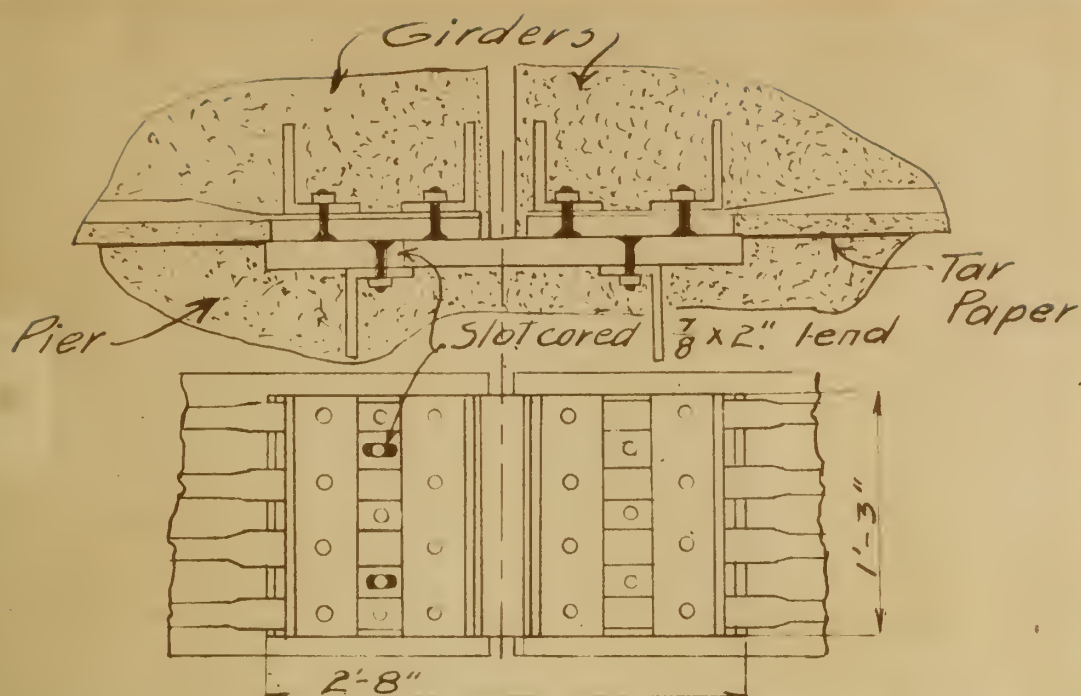


Fig. 26. Expansion Joint And Girder Seat.

upon cast iron, and the stress necessary to produce slipping is:

$$00.2 \times 75,750 = 15,150.0 \text{ pounds.}$$

The steel area at the center of the beam where the steel is already stressed to the allowable flectural limit is 22.04 square inches. The additional stress caused by the temperature is:

$$\frac{15,150}{22.04} = 690 \text{ pounds per square inch,}$$

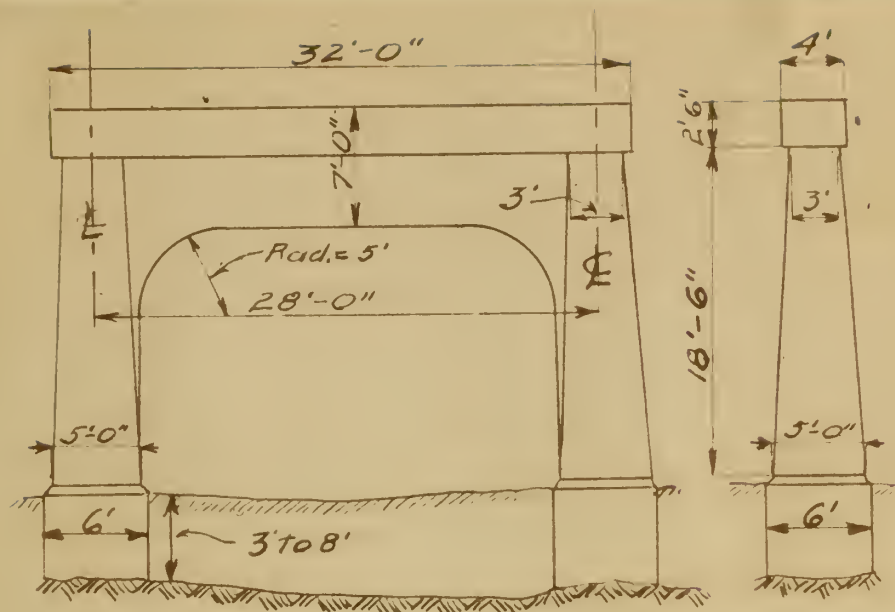
which is less than 2,000 pounds additional unit stress as allowed for in Table I as the probable magnitude of the temperature stress.

The columns of the piers should be designed to give a pleasing effect and correct proportion in addition to efficiency.

Fig. 27 shows the dimensions of the columns as taken.







*Fig. 27. Dimensions of Columns.*

The total load brought by the floor system upon each pier is:

From Intermediate Girders  $3 \times 151,500 = 454,500$  pounds.

" Side Girders  $2 \times 150,400 = 300,800$  pounds.

Total  $= 755,300$  pounds.

The weight of the cross beam is  $14 \times 3,523 = 49,300$  pounds, which makes a grand total of:

Street and Walks 755,300 pounds.

Cross-girder 49,300 pounds.

Total 804,600 pounds.

If a net section of 36 x 36 square inches be assumed to carry this load, the unit pressure is:

$$\frac{804,600}{36 \times 36} = 620 \text{ pounds per square inch.}$$



As the allowable flectural unit stress is 600 pounds per square inch, the excess of 20 pounds will be permitted as it is a result of direct compression. The pressure on the footings will be in addition to the weights above, the weight of the column itself. This when computed is found to be 53,600 pounds, and the total weight is therefore:

$$804,600 + 53,600 = 858,200 \text{ pounds,}$$

which is distributed over an area 6 feet square. The unit pressure upon the limestone upon which these piers rest is:

$$\begin{aligned} P &= \frac{858,200}{36} \\ &= 23,800 \text{ pounds per square foot,} \\ &= 11.9 \text{ tons per square foot.} \end{aligned}$$

The vertical rods which are placed near the face of the pier in order to prevent surface cracks will be carried to within 6 inches of the limestone. At this point 6 inch bends will be hooked under a horizontal mat of 3/4-inch rods placed on 8-inch centers in both directions, and extending over the base of the footing, thus tending to equalize the pressures.

#### VIII. DESIGN OF ABUTMENTS.

The north abutment of the viaduct, shown in Fig. 5, will now be designed. The counterforts are spaced on 7-foot centers, thus corresponding to the spacing of the floor system girders which will be supported by them.

The pressure back of the abutment is taken as that



resulting from a liquid of 25 pounds per cubic foot in weight. This assumption is recommended in the text on p. 373. The total height of the abutment at the north end of the structure will be about 22 feet after the approach is graded. The total pressure upon a vertical strip of wall one foot wide is, using the above assumption,

$$H = \frac{25}{2} \times 22 \times 22$$

$$= 6,050 \text{ pounds.}$$

The total horizontal pressure per panel length of 7 feet is:

$$7 \times 6,050 = 42,350 \text{ pounds,}$$

and the unit pressure at the foot of the abutment is:

$$P = 25 \times 22$$

$$= 550 \text{ pounds per square foot.}$$

The distance between counter forts, allowing each a width of 2-feet, is  $(7-2)=5$  feet, and the moment which a horizontal section of curtain 1-foot wide must carry is:

$$M = \frac{550 \times 5 \times 5 \times 12}{8}$$

$$= 20,600 \text{ pound-inches.}$$

The continuity of the curtain past the counter forts will be neglected, assuming the coefficient of  $1/8$  rather than  $1/10$  as in the previous design for continuous spans.

The practical minimum thickness of a wall of this class is 18 inches; and the stresses from the above moment upon a slab of this thickness and 1-foot in width will now be found.





The modulus of rupture for this section is, using the effective depth as 17 inches,

$$R = \frac{20,600}{12 \times 17 \times 17}$$

$$= 6.7$$

Since this value of R is below any given in the diagram, it is evident that the stress from the moment will be negligible. However, it must be reinforced near each face to prevent surface cracks. These rods will be 3/8-inch in diameter on 8-inch centers vertically and horizontally.

The maximum end reaction of one girder for a 47-foot span as already found is 75,750 pounds. Since the span adjoining the abutment is only 35 feet, the girder reaction will be,

$$\frac{35}{47} \times 75,750 \text{ pounds} = 56,400 \text{ pounds.}$$

The weight of the concrete in one panel length of 7 feet of the abutment as shown in Fig. 28 is 76,250 pounds. This combined with the above floor beam reaction gives:

Floor beam	56,400 pounds
One panel abutment	<u>76,250 pounds.</u>
Grand total	132,650 pounds.

By the principles of mechanics the resultant of these two forces is found to be 9.52 feet from the toe of the abutment. Combining the above weight of 132,650 pounds with the horizontal force of 42,350 pounds, resulting from earth pressure as already computed, the resultant, by means of a graphical solution, is



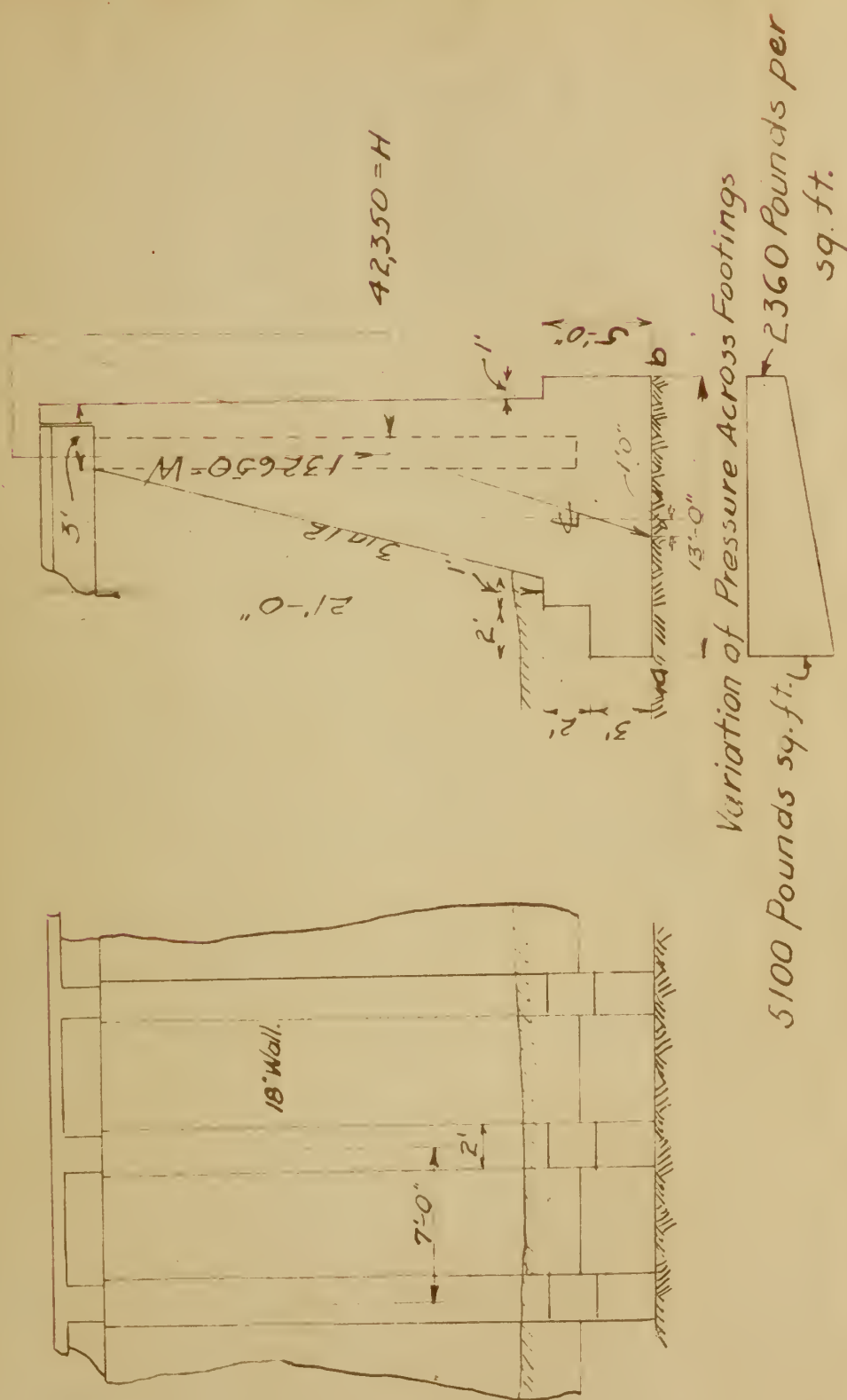


Fig. 28. Elevation and Section of North Abutment.



found to pierce the base at a distance of one foot from the center of the footing. See Fig. 28.

The maximum pressure resulting from this eccentricity will now be computed from the formulae:

$$P = \frac{W}{l} \pm \frac{6Wd}{l}$$

where

$P$  = the unit pressure per square foot,

$W$  = the load per foot of footing measured perpendicular to the overturning force,

$l$  = the length of the footing measured in the direction of the overturning force, and

$d$  = the eccentricity of the resultant.

Before substituting in this formulae the total weight as found must be divided by 2 since the counterfort is 2 feet wide. Using the value of  $W$  as  $\frac{132,650}{2}$  and  $d$  and  $l$  as one and 13 feet respectively, the pressures at  $a$  and  $b$ , Fig. 27.

$$\begin{aligned} P &= \frac{66,325}{13} \pm \frac{6 \times 66,325 \times 1.00}{13 \times 13} \\ &= 5,100 \pm 2,360 \text{ pounds per square foot.} \end{aligned}$$

The pressure at the toe of the counterfort is then ,

$$5,100 + 2,360 = 7,460 \text{ pounds per square foot,}$$

and the minimum unit pressure at the heel is,

$$5,100 - 2,360 = 2,740 \text{ pounds.}$$

These are far on the side of safety as the abutment rests upon





the limestone underlying the surface of the ground.

The total pressure transferred to the abutment at each girder seat is, as already computed from a 35-foot span, 56,400 pounds. The area of the bearing surface is, as in the main girders, 13 x 15 inches, and this gives a unit pressure of:

$$P = \frac{56,400}{13 \times 15}$$

= 289 pounds per square inch.

Since the cross section of the counterfort is at every point greater than the area of the above plate the unit pressure in the concrete will be less than the above value of 289 pounds, thus giving a probable factor for compression of between two and three.

To equalize the pressure across the footing, 3/4-inch rods will be placed on 4-inch centers both ways across the base, 6-inches from the limestone. The 3/8-inch rods which are placed near the face of the counterforts to prevent surface cracks will be hooked under this mat. They will be spaced on 8-inch centers both horizontally and vertically.

The south abutment is of the same general dimensions as the north. However, the wings will be turned back at right angles to the abutment instead of being straight, thus increasing the stability of the abutment.

The concentrated load from the floor system at each girder seat will, however, both be that of a 47-foot span instead of a 35-foot span as at the north abutment.



Referring to Fig. 27, the value of  $W$  for the south abutment will be  $75,750 + 76,250 = 152,000$  pounds. Since this is greater than the value of 132,650 as used in the graphical solution for the other abutment, the resultant will fall nearer the center and thus produce less eccentricity. Using  $W$  in the formula as the above value divided by two to secure pressure in units, and assuming an eccentricity of 1 foot, as in the other case, the pressures at the toe and heel of the counterforts are found to be 8,550 and 3,150 pounds per square foot respectively. As these are not excessive pressures upon a limestone ledge the design which was made for the other abutment will be suitable for that at the south of the viaduct.

The wings of the abutment will be designed as a retaining wall in the next article.

#### IX. DESIGN OF RETAINING WALLS.

The retaining walls for the south approach vary in height from 2 to 20 feet. The sections of three different height walls will now be designed, and from these the intermediate sections can be interpolated.

The method of analysis is similar to that used in the design of the abutments. The earth pressure is taken as equivalent to that resulting from a liquid weighing 25 pounds per cubic foot. In addition to this certain other assumptions will be made. Since the retaining wall rests directly upon an earth foundation the weight of the curtain wall is assumed to be



carried by its own base, thus leaving the weight of the concrete in the counterfort to counteract the overturning effect of the earth upon the curtain wall. Furthermore, the weight of the earth back of the counterfort will be neglected.

Fig. 29 shows cross-sections of the three heights of wall for which computations are made. The counterforts are 4 feet in depth at the top to provide for a walk which will be placed along the wall after the earth-fill is made. The projection down from the heel of the counterfort is intended to resist any tendency of the counterfort to slide. For the sake of uniformity the projecting web of the counterfort will be made 2 feet in width throughout.

The thickness of the curtain wall is taken as 18 inches. The maximum moment which a strip of wall of this thickness will have to carry will be at a depth of 18 feet between two 5 foot counterforts. At this depth the pressure is  $18 \times 25 = 450$  pounds per square foot. The panel lengths are taken as 11 feet 9 inches to correspond to the railing panel lengths. The length of the slab under consideration is then  $11.75 - 5.0 = 6$  feet 9 inches long, and the moment is:

$$M = \frac{6.75 \times 6.75 \times 12 \times 450}{8}$$

$$= 30,810 \text{ pound-inches.}$$

From this, R, the section modulus, is found to be:

$$R = \frac{30,810}{12 \times 18 \times 18}$$

$$= 7.9$$





# All 11'-3" Panels.

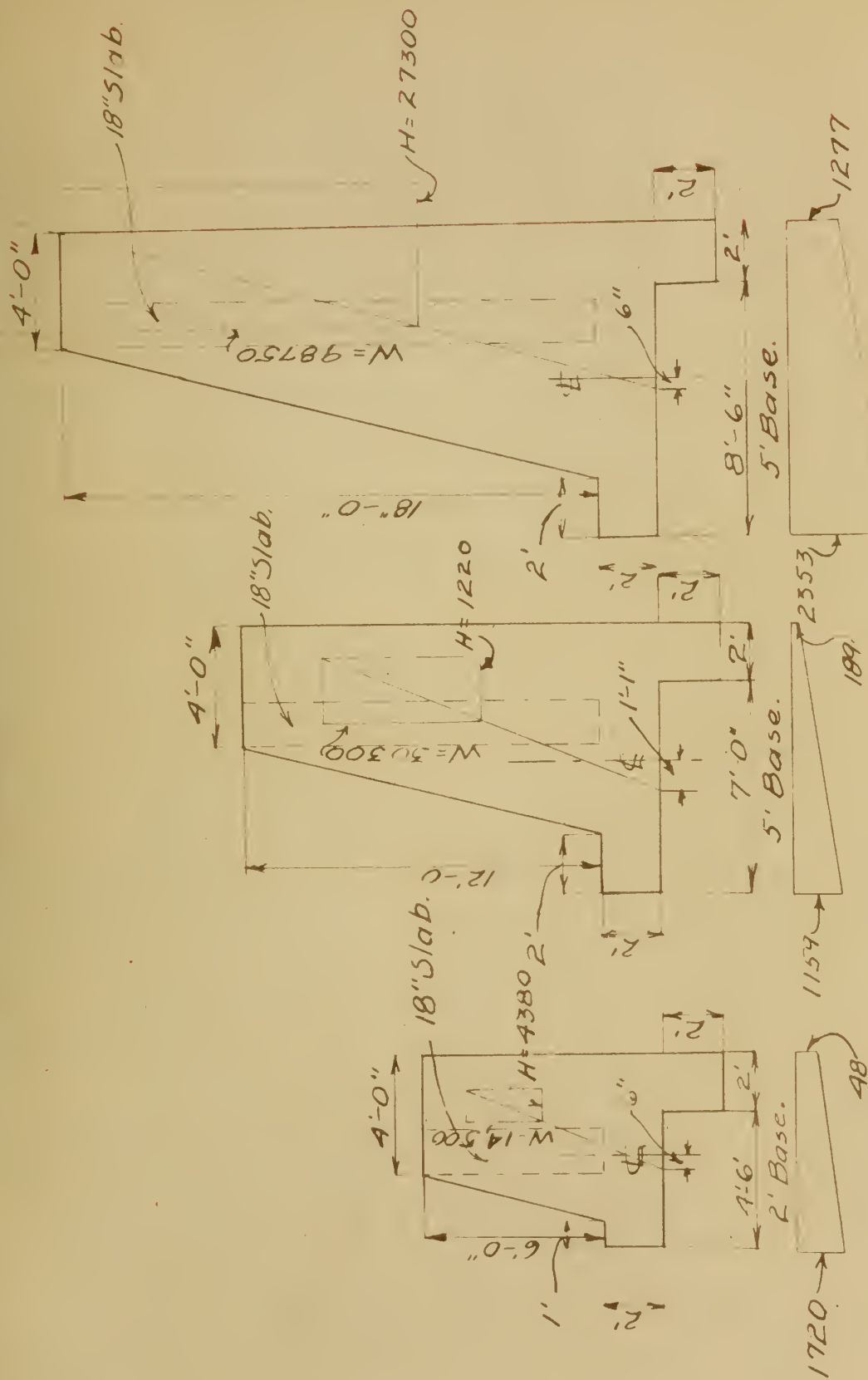


Fig. 29. Cross Section of Retaining Walls and Variation of Pressure Across Base.



Entering the table with this value of R and a reinforcement of 0.1 percent which the rods placed near the face to prevent surface cracks will supply, the unit stresses in the steel and concrete are found to be 6,000 and 150 pounds per square inch. Inasmuch as these are below the allowable unit stress, no additional reinforcement will be necessary.

Based upon the above assumptions regarding earth pressures and resisting forces, the resultants for each height of wall have been found to pierce the base of their respective footings as shown in Fig. 29. The variation of pressure due to eccentricity across the sections A-A is computed from the formulae:

$$P = \frac{W}{L} \pm \frac{6Wd}{L^2}$$

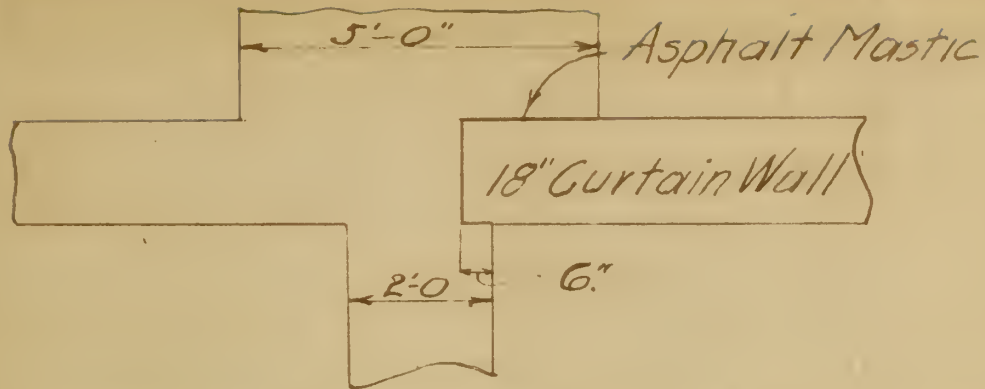
as given in the design of the abutments.

Using the above formulae, the values plotted in Fig. 29, are found to be the unit values of the pressures in pounds per square foot across each of the bases. The maximum pressure, 2,353 pounds per square foot, is a safe value for the bearing upon the earth.

To relieve any stresses set up in the wall due to expansion or settlement of footings, a joint will be placed in the wall every 47 feet. A horizontal section of this joint is shown in Fig. 30.

The slab reinforcement will consist of 3/8-inch rods on 8 inch centers vertically and horizontally. These rods placed near the surface of the slab to prevent cracks. This





*Fig. 30. Horizontal Section Through Expansion Joint in Retaining Wall.*

reinforcement will also be used in the counterforts, the lower ends of the vertical rods being hooked under a mat of 3/4-inch rods on 4-inch centers.

#### X. PAVEMENT

Inasmuch as a pavement possessing as great an imperviousness as possible is desirable to prevent seepage through to the concrete below, several precautions should be taken.

The blocks themselves should conform to standard specifications and be thoroughly creosoted. The filler should be of asphaltic mastic, which in addition to making a water-tight joint between the blocks will provide for any expansion or movement due to temperature changes. The movement of the floor system proper will be taken care of by the plates as shown in Fig. 3.





## XI. WATER PROOFING AND WEEPHOLES.

To prevent any water which may seep through the pavement from coming in contact with the reinforcing steel in the floor slab, the top of the concrete is given a layer of waterproofing. This will consist of a double layer of asphaltic mastic and burlap, the latter applied alternately with the hot mastic painted directly upon slab surface and sides of the gutters. The weep holes are shown in Fig. 6.

## XII. ESTIMATE OF COST.

The cost of the structure can be estimated only from the costs of previous structures. In the Engineering News for February 10, 1910, A. N. Johnson gives some data which will be used in making an estimate. The data as furnished for spans of 40 to 50-feet in length which correspond to the 47-foot span of the design vary from \$11.43 to \$13.71 per cubic yard. Mr. Johnson states that work of this general nature would be of little profit unless it was paid for at a price of \$10 to \$12 per cubic yard. From this statement together with the above costs as quoted it would seem that \$12 per cubic yard would be a reasonable and fair cost for the floor system and piers.

The total yardage is computed to be 5,122 for the viaduct, omitting the retaining walls for the south approach. This at \$12 per cubic yard gives a cost of \$69,464.00 for the viaduct proper or \$3.47 per square foot of roadway. The cost



per square foot of the Mulberry Street Viaduct of Philadelphia was \$2.64 per square foot. However, as this was a larger structure and of the arch type it is reasonable to expect a higher cost in the viaduct just designed.

The cost of the retaining wall will be less per cubic yard than that of the floor system. The cost of a plain retaining wall is about \$8 per cubic yard. Using this as a unit cost the computed yardage as 634, the cost of this portion of the viaduct is \$2,853.00, making a grand total cost of \$72,317.00 for the entire viaduct.









# PLATE V

Note All wall & pier faces reinforced with  $\frac{3}{4}$ " rods on 8" centers vertically and horizontally. Vertical rods hooked under footing reinforcement.

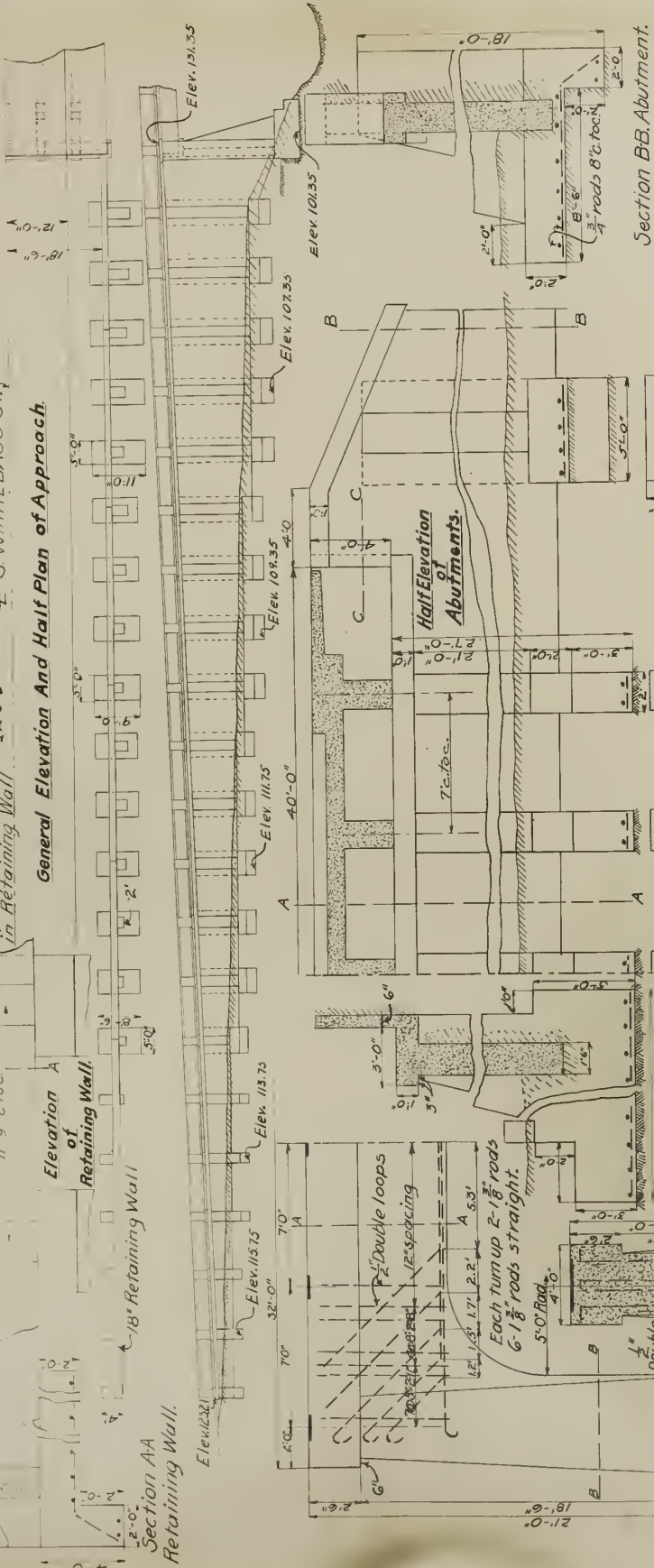
6" recess painted with Asphalt Mastic.

Enlarged Horizontal Section BB of Expansion Joint in Retaining Wall

Elevation of Retaining Wall

Section AA Retaining Wall

## General Elevation And Half Plan of Approach.



### Section AA Abutment.

### Section A-A Pier.

### Section BB Pier.

### Half Elevation of Pier.

### Section BB Abutment.

DESIGN OF A  
REINFORCED CONCRETE VIADUCT  
FOR THE  
CITY OF ROCKFORD.  
BY  
Geo. Harold Smith

### Section C-C Turnback South Abutment









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